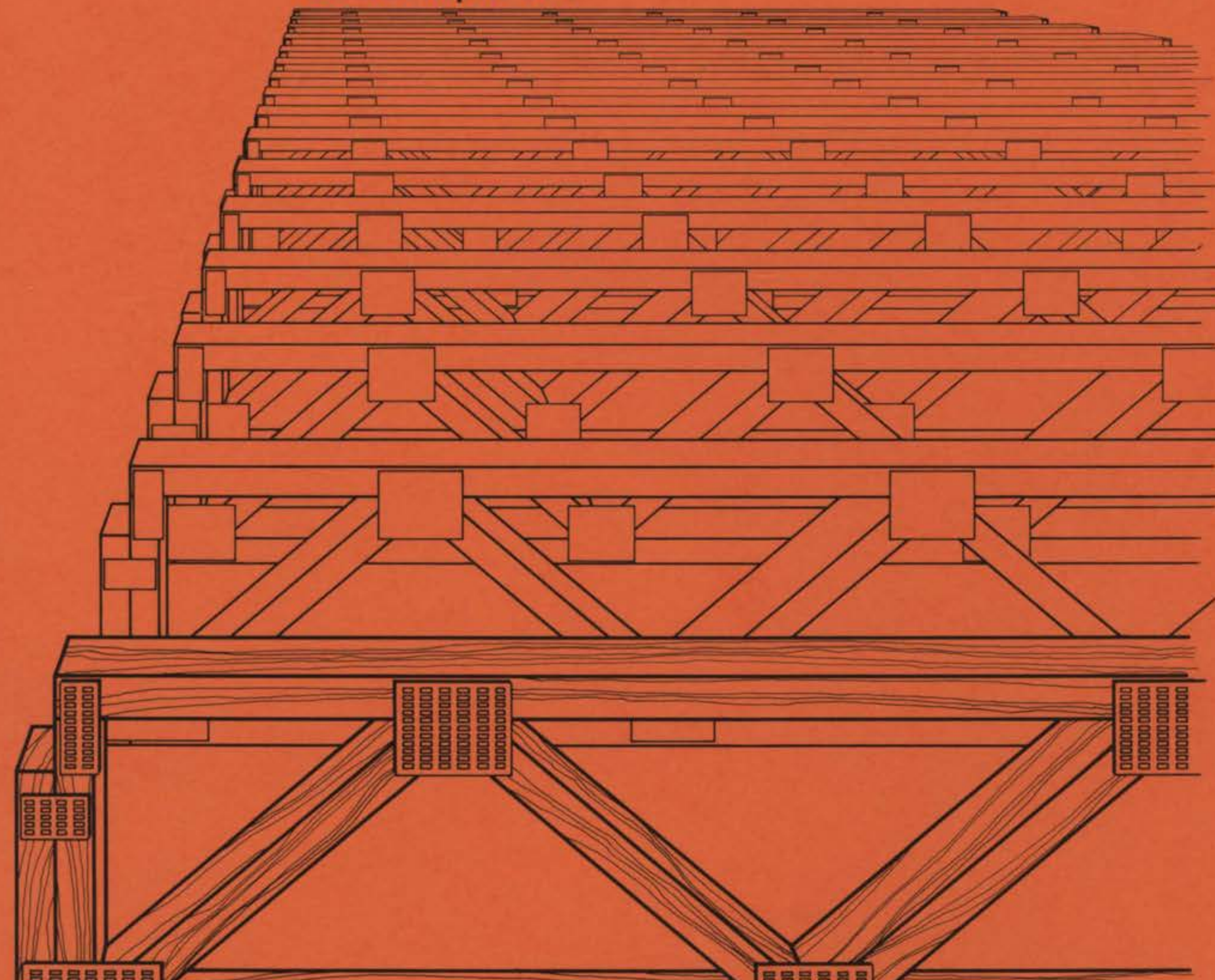
TESTING AND ANALYSIS OF 4 x 2 PARALLEL-CHORD METAL-PLATE-CONNECTED TRUSSES

Research Report 81-1



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I. INTRODUCTION

The objectives of this research were to provide experimental data and analytical results as input information for use in design specifications for metal-plate-connected, parallel-chord floor truss designs.

The Truss Plate Institute (TPI) has developed a design specification, Parallel Chord Truss Design Criteria, PCT-80 (15).[1] This study presents an investigation of the precision of PCT-80 as a practical engineering tool for the design of 4 x 2 parallel-chord trusses.

The principal portions of the experimental data are correlated by analyses using the Purdue Plane Structures Analyzer (PPSA) (11) in order to develop the parameters of an adequate structural model. The model is then used to appraise proposed applied design techniques.

1. Numbers in parentheses refer to the reference list.

Table 1.1. Truss member data: Grade book modulus of elasticity (MOE) values; MOE from flatwise concentrated load on 10-foot span; specific gravity, G, at oven-dry weight and green volume; and moisture content, MC, in percent at time of test. These trusses had been subjected to short-term tests prior to determination of the tabulated values.

Grade MOE (10^6 psi) UC/LC	Upper MOE* Limit UC/LC	Lower MOE* Limit UC/LC	Truss Number	Chord	Chord MOE (10^6 psi)	Chord MC percent	Chord G	Webs G (avg.)
			1.1	UC	2.05	11.5	.48	.47
				LC	2.39	11.8	.53	
1.90/1.90	2.83/2.83	.97/.97	1.2	UC	1.67	8.7	.42	.54
				LC	2.07	11.9	.60	
			1.3	UC	2.00	10.3	.54	.51
				LC	2.69	11.0	.54	
			2.1	UC	2.29	6.3	.55	.47
				LC	2.15	6.0	.54	
1.60/1.70	2.38/2.53	.82/.87	2.2	UC	2.94	6.3	.45	.44
				LC	1.81	6.7	.43	
			2.3	UC	2.77	6.5	.58	.46
				LC	1.84	6.5	.54	
			3.1	UC	2.13	12.0	.42	.47
				LC	2.33	12.0	.54	
1.70/1.70	2.53/2.53	.87/.87	3.2	UC	1.53	12.0	.44	.42
				LC	1.45	12.0	.56	
			3.3	UC	1.22	12.0	.41	.48
				LC	1.23	12.0	.44	
			4.1	UC	1.77	6.9	.37	.53
				LC	2.20	7.3	.43	
1.90/1.90	2.83/2.83	.97/.97	4.2	UC	1.95	7.1	.52	.41
				LC	2.14	7.4	.44	
			4.3	UC	2.12	7.4	.43	.42
				LC	2.16	7.3	.49	
			5.1	UC	2.72	11.7	.51	.48
				LC	2.15	11.6	.46	
1.90/1.90	2.83/2.83	.97/.97	5.2	UC	2.90	11.1	.58	.47
				LC	2.80	9.9	.58	
			5.3	UC	3.33	11.4	.60	.50
				LC	3.25	10.2	.56	
			6.1	UC	1.93	7.0	.57	.49
				LC	1.97	7.0	.39	
1.90/1.90	2.83/2.83	.97/.97	6.2	UC	1.86	7.0	.56	.48
				LC	2.10	7.0	.41	
			6.3	UC	1.93	7.0	.46	.45
				LC	2.01	7.0	.50	

* Using a coefficient of variation of .25 as given in the NDS (2) and assuming a normal distribution, 2.5 percent of all sampled MOE values are expected to lie above the upper limit and 2.5 percent below the lower limits. The grade MOE value has been taken as the average of the assumed normal distribution.

Table 1.2. Connector plate thickness measurements from six randomly selected plates from each of the trusses subjected to short-term testing.

Truss No.	Thickness (in.)						Average (in.)
1.1	.037	.037	.037	.038	.037	.037	.037
1.2	.037	.037	.038	.038	.036	.037	.037
1.3	.037	.039	.037	.037	.037	.037	.037
2.1	.042	.041	.040	.041	.043	.044	.042
2.2	.041	.041	.043	.042	.042	.043	.042
2.3	.043	.043	.041	.041	.043	.041	.042
3.1	.040	.040	.060*	.060*	.060*	.040	.040
3.2	.040	.040	.060*	.060*	.040	.039	.040
3.3	.040	.038	.038	.040	.057*	.057*	.039
4.1	.037	.038	.038	.039	.036	.037	.038
4.2	.036	.037	.037	.037	.037	.036	.037
4.3	.037	.037	.037	.037	.037	.036	.037
5.1	.038	.037	.039	.039	.038	.038	.038
5.2	.040	.040	.040	.040	.039	.038	.040
5.3	.039	.039	.037	.038	.040	.040	.039
6.1	.039	.038	.038	.040	.039	.039	.039
6.2	.040	.040	.040	.040	.038	.039	.039
6.3	.038	.038	.038	.040	.039	.040	.039

* Chord splice joints, .059 in. avg.

SPONSORS AND COOPERATORS

Strong interest for this study has been exhibited by the Federal Housing Administration (FHA) because of the increased use of parallel-chord floor trusses. This cooperative study involved the Small Homes Council-Building Research Council, University of Illinois (SHC-BRC-UIUC); Wood Research Laboratory, Purdue University (WRL); and the Department of Housing and Urban Development (HUD), who partially financed the project. The work was performed at both universities with additional financial aid from TPI and by the universities' shares made possible through integrating the study with existing programs.

HISTORY

The original program encompassed five separate designs of parallel-chord floor trusses furnished by members of TPI. The number of designs was later changed to six.

The first draft of the research proposal evolved in the spring of 1976 after several meetings with the technical advisory committee of TPI. This draft was submitted to FHA for review and comments. From the various discussions and review comments, the study plan was finalized and activated by the two universities, TPI, FHA, and HUD. Testing began in the summer of 1976.

TEST TRUSSES AND MATERIALS

The plan was to receive test units from five plate manufacturers, conduct short-term load-deflection and load-to-failure tests on three units of each design, long-term load tests on two additional units of each design, and test tension joint specimens using lumber from the upper and lower chords of the short-term units. The truss designs received for study are shown in Figure 1.1.

Five TPI member companies offered to furnish test units – Truswal, Hydro-Air, Lumbermate, Automated Building Components (ABC), and Troy Truss. Before completion of the short-term testing, Troy Truss went out of business and their units were eliminated from the study. The Troy units were replaced with test units from Truswal and Alpine having off-center rectangular openings. These appear as truss types 5 and 6, respectively, in Figure 1.1.

Upon completion of each short-term load test, the truss was disassembled and each top and bottom chord was subjected to modulus of elasticity and specific gravity measurements. Several web members were also collected to determine specific gravity. These measurements are shown in Table 1.1, along with moisture content of the chords at time of test.

The trusses all appeared to be good to excellent in manufacturing quality. Measurements of MOE showed the lumber to be above average, with 30 of the 36 chords higher than the book grade value and with five of these 30 having exceptionally high values (Table 1.1). This latter judgement is based on fitting a normal curve to the visual grade E-values and labeling MOE values that lie above the 97.5 percentile as exceptional. In terms of specific gravity the chord lumber also may be judged as generally above average. The National Design Specification (NDS) (8) value of specific gravity is listed as .55 for Southern Pine based on oven-dry weight and oven-dry volume. This value converts to .48 on the basis of oven-dry weight and green volume to correspond with the values reported in Table 1.1. Consulting the latter table, it is noted that 21 of the 36 chords were at or above .48 in specific gravity. The web specific gravity averages reported in Table 1.1 show 8 out of 18 values at or above the value .48 and ranged from .41 to .54. Fitting a normal distribution to the average of .48 and using the coeficient of variation .10 reported in the Wood Handbook (16) yields a 2.5 percentile of .39 and a 97.5 percentile of .57. The webs as a whole would be judged as near an expected average on the basis of these figures.

Plates varied some in thickness among the truss type groups but were quite consistent within any truss type. Sampling of plate thickness is reported in Table 1.2.

TEST FACILITIES

The short-term tests were conducted at SHC-BRC-UIUC and the long-term tests were conducted in a rented building in Brookston, Indiana, 12 miles north of Purdue University. Metal plate joint tests, also included in this report, were conducted at the Wood Research Laboratory, Purdue University.

Figure 1.1. Truss designs tested in the projects. Actual dimensions taken from the test units.

TRUSS TYPE 1

FIGURE Ia

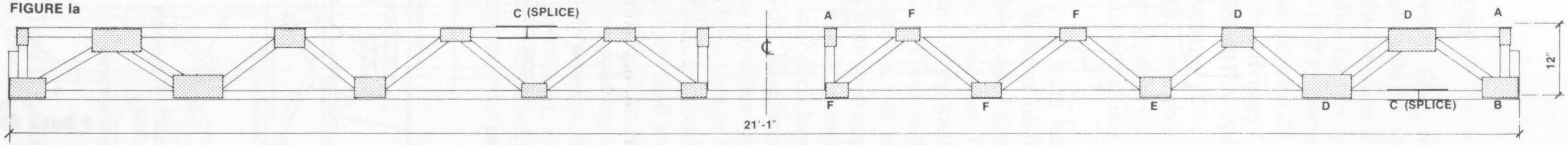

PLATE SIZES
- A: 2" x 3"
- B: 6" x 3-1/2"
- C: 10-1/8" x 3"
- D: 8" x 4"
- E: 5" x 3-1/2"
- F: 4" x 2-1/2"

LUMBER

TOP: SPIB NO. 1 DENSE, KD 2250F ,(344), SYP;
SPIB NO. 1 DENSE, KD 2250F, (700), SYP,
BOTTOM: SPIB NO. 1 DENSE, KD 2250F, (720), SYP,
WEBS: SPIB NO. 1 DENSE, KD 2250F, (700), SYP

DESIGN DATA

TOP CHORD L.L.: 40 PSF
TOP CHORD D.L.: 10 PSF
BOTTOM CHORD D.L.: 5 PSF

TRUSS TYPE 2

FIGURE Ib

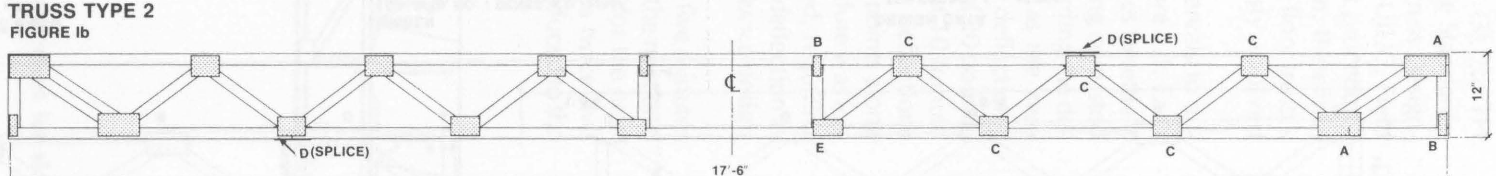

PLATE SIZES
- A: 6" x 3-1/2"
- B: 1" x 3"
- C: 4" x 3"
- D: 5" x 3"
- E: 4" x 2-1/2"

LUMBER

TOP: SPIB NO. 2 DENSE, SYP,
BOTTOM: SPIB NO. 2 DENSE, KD I850F, (403), SYP,
WEBS: SYP, NO VISIBLE GRADE MARKS

DESIGN DATA

TOP CHORD L.L.: 40 PSF
TOP CHORD D.L.: 10 PSF
BOTTOM CHORD D.L.: 10 PSF

TRUSS TYPE 3

FIGURE Ic

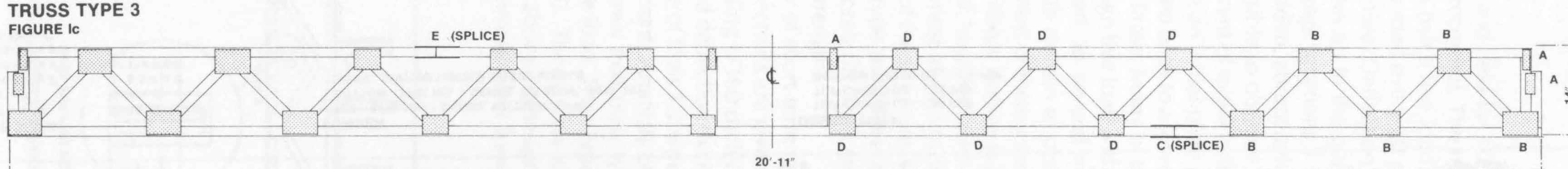

PLATE SIZES
- A: 1-1/2" x 3-1/2"
- B: 5-1/4" x 4"
- C: 8" x 3"
- D: 4" x 3-1/2"
- E: 6-3/4" x 3"

LUMBER

TOP: SPIB NO. 2 DENSE, KD 1850F, (73), SYP,
BOTTOM: SPIB NO. 2 DENSE, KD I850F, (73), SYP,
WEBS: SPIB NO. 2 DENSE, KD I850F, (73), SYP

DESIGN DATA

TOP CHORD L.L. = 40 PSF
TOP CHORD D.L. = 10 PSF
BOTTOM CHORD D.L. = 5 PSF

Figure 1.1 (continued)

TRUSS TYPE 4
FIGURE Id

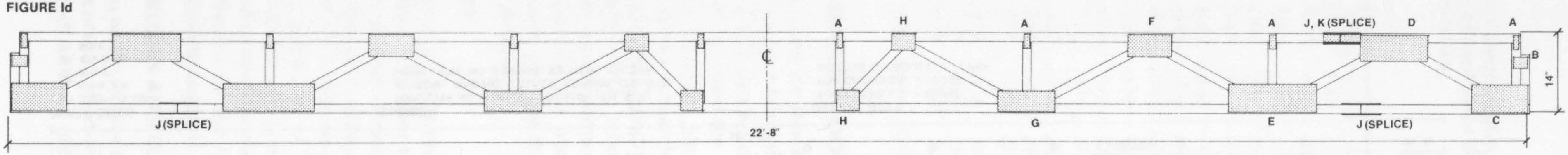

PLATE SIZES
A: 1" x 2-5/8"
B: 3" x 2"
C: 10-1/8" x 5"
D: 12-1/4" x 5"
E: 15-3/4" x 5"
F: 7-7/8" x 4"
G: 10-1/8" x 4"
H: 4" x 3"
J: 6-3/4" x 3"
K: 1" x 6-3/4"

LUMBER
TOP: SPIB NO. 1 DENSE, KD 2250F, SYP,
BOTTOM: SPIB NO. 1 DENSE, KD 2250F, (870), SYP,
WEBS: SYP, NO VISIBLE GRADE MARKS

DESIGN DATA
TOP CHORD L.L. = 40 PSF
TOP CHORD D.L. = 10 PSF
BOTTOM CHORD D.L. = 5 PSF

TRUSS TYPE 5
FIGURE Ie

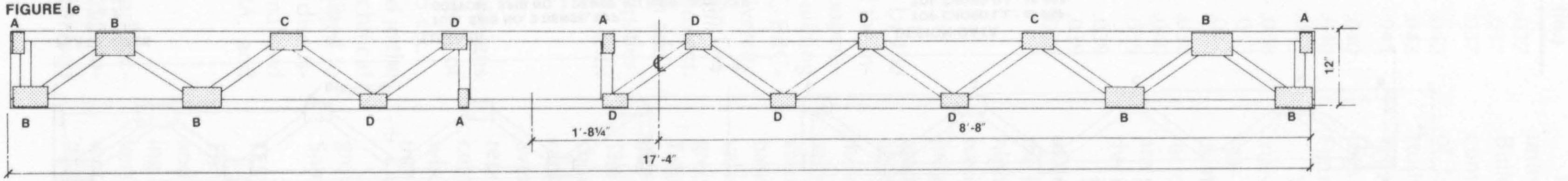

PLATE SIZES
A: 2" x 3"
B: 6" x 3½"
C: 5" x 2½"
D: 4" x 2½"

LUMBER
TOP: SPIB, NO. 1 DENSE, K.D., 2250F,
BOTTOM: SPIB, NO. 1 DENSE, K.D., 2250F,
WEBS: SYP, NO VISIBLE GRADE MARKS

DESIGN DATA
TOP CHORD L.L. = 40 PSF
TOP CHORD D.L. = 10 PSF
BOTTOM CHORD D.L. = 5 PSF

TRUSS TYPE 6
FIGURE 1f

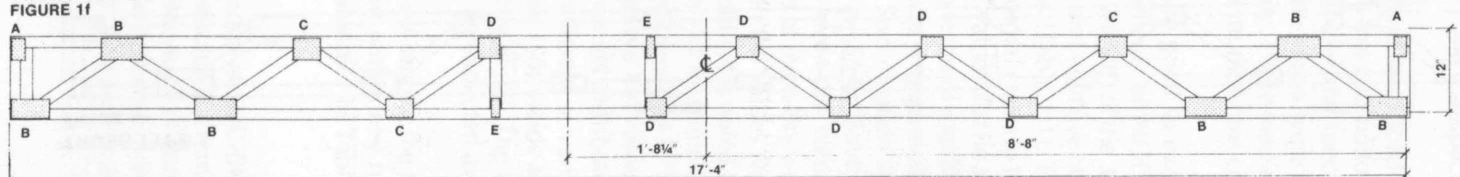

PLATE SIZES
A: 2" x 3"
B: 6" x 3"
C: 4" x 3"
D: 3" x 3"
E: 1" x 3"

LUMBER
TOP: TPI NO. 1 DENSE, S-DRY, (335), SYP, 2000F,
BOTTOM: TPI NO. 1 DENSE, S-DRY, (335), SYP, 2000F,
WEBS: SPIB NO. 3, KD, (406), SYP

DESIGN DATA
TOP CHORD L.L.: 40 PSF
TOP CHORD D.L.: 10 PSF
BOTTOM CHORD D.L.: 5 PSF

II. SHORT-TERM TESTS

PROCEDURE

At the present time, no uniformly accepted standard exists applicable to testing parallel-chord trusses. A survey of the literature was made to establish a test procedure suitable for this type of structural component: (a) ASTM E6-11-73, Testing Truss Assemblies (3), (b) TPI Design Specification-74 (13), (c) Performance Standards, Joint Industry Advisory Committee on Roof Truss Design-1963 (6), (d) HHFA-1948 (5), (e) SHC-BRC-UIUC, Field Testing Trusses, 1958 (7). None of these test procedures was directly applicable without modification; therefore, a specific procedure was developed for these floor trusses incorporating features from the previously evolved methods.

The trusses were loaded at one foot intervals to approximate uniform loading as shown in Figure 2.1. Each truss was pre-loaded to design load three times to settle it into the test apparatus. The first stage of testing consisted of application of the total dead load and recording the deflection. Deflection at dead load was used as the zero reference point for all subsequent live-load deflections. Live loads were then applied in increments of 20 pounds per linear foot (10 pounds per square foot) at 10 minute intervals. After each load level was reached, deflections were recorded. Readings were taken at three points along the bottom chord (see Figure 2.1). This procedure was followed until full design live load was reached, requiring about one hour to complete. After recording deflection at full live load, the load was then increased to twice design live load and held for three hours.

Deflection readings were recorded every five minutes for the first 25 minutes, every 10 minutes for the next one hour and 50 minutes, and every 15 minutes for the next 45 minutes. After holding this dead load plus twice live load for the three-hour period, the load was reduced to the full dead load and held for 10 minutes. Residual deflection was then recorded. The truss was again reloaded to dead load plus twice live load; then deflection observations were recorded every 10 psf as the load was further increased to failure. Deflection was read at the centerline scale in all cases up to the point of failure and, in some cases, also at gage locations 1 and/or 3. Using hydraulic equipment, sudden, abrupt failures were not experienced and it was possible to observe the behavior of the structure after the event of initial failure. It is interesting to note that even when an initial plate peel failure occurred, and the load dropped back to a lower level, the trusses did not completely collapse. Many of them continued to carry at least twice design live load with one or more connections completely apart. In several instances, the lower loads were allowed to remain to observe creep or continuing deflection. In most of these cases the trusses continued to carry a load without further failures occurring. It should be remembered, however, that a gravity load behaves differently and almost always produces a sudden and complete collapse of the structure, which means that hydraulic tests of the type used here are technically interesting but do not necessarily indicate that trusses will always have reserve strength.

A summary of short-term test results is given in Table 2.1. Three deflections are given for each truss along with the corresponding L/360 deflection criteria. Trusses 1.2, 1.3, and 3.3 had deflection exceeding the L/360 value but only in the case of truss 1.2 would this be considered as a significant amount. Analysis of this truss, given later in this report, shows that truss type 1 was slightly underdesigned for the load. Truss types 2, 5, and 6 deflected far less than L/360. These were shorter spans and all had chord E-values above the design table values. In fact, truss 5.3 had uncommonly stiff lumber in both chords.

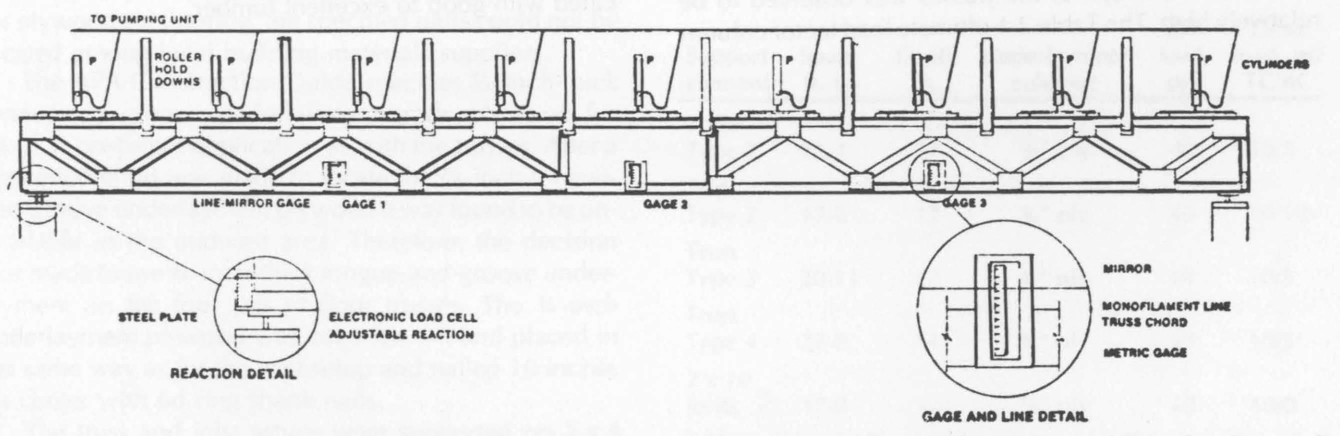

Figure 2.1. Typical loading and deflection arrangement for short-term test set-up. Loading points and gage location varied according to truss dimensions (see Figure 1.1).

Table 2.1. Summary of short-term test results.

Truss Identification	Design Load, psf DL + LL	Out-to-Out Span, L ft.-in.	Depth in.	Allowable L/360 in.[1]	Deflection at LL, in. (Center) Gage 1	Gage 2	Gage 3	Ult. Load Factor, K[2]	Failure Type[3]
1.1					.45	.67	.48	2.70	P
1.2	15 + 40	21-1	12	.68	.57	.86	.59	2.76	P
1.3					.48	.70	.48	2.82	P
2.1					.23	.31	.24	2.57	P
2.2	20 + 40	17-6	12	.56	.26	.29	.21	2.69	P
2.3					.24	.33	.24	2.61	P
3.1					.35	.51	.37	2.84	P
3.2	15 + 40	20-11	14	.68	.44	.63	.44	3.14	PW
3.3					.47	.70	.48	3.13	W
4.1					.55	.74	.48	3.46	B
4.2	15 + 40	22-8	14	.74	.49	.71	.50	3.52	B
4.3					.45	.64	.43	3.62	B
5.1					.24	.35	.34	2.74	P
5.2	15 + 40	17-4	12	.56	.15	.30	.20	2.40	P
5.3					.19	.28	.15	2.92	P
6.1					.24	.38	.21	4.38	P
6.2	15 + 40	17-4	12	.56	.24	.36	.22	4.94	PW
6.3					.24	.36	.20	4.75	P

[1]L = Clear span.
[2]Ultimate Load = Dead Load + K (Live Load).
[3]W = Wood; P = Plate Peel; PW = Plate Peel with Wood; B = Plate Buckle and Tear.

The complete load-deflection test records are given for each short-term test in Appendix B. The first chart indicates the sequence of loading and points out the features to be observed. These characteristics pertain to all subsequent records in the Appendix.

The column showing types of failure in Table 2.1 indicates that plate-associated failures were predominant. Wood failures occurred in only three cases. Truss 3.2 had a shear failure under a lower chord splice in a place parallel to and just beneath the plate. Truss 6.2 had two similar type failures, one at a heel plate and the other at the outer connection of the first tension web from the same heel end. Truss 3.3 experienced a failure (probably initiated at a knot in the lower chord) at a location of 1/8 span off the center of the truss span.

The test strength of the trusses was observed to be relatively high. The Table 2.1 ultimate load factor column gives the multiple of live load carried at failure, in addition to the dead load. All values of the live-load factor were above the commonly acceptable range of 2¼ to 2½. One truss carried 4.94 times live load plus dead load at failure. This is far beyond traditional targets for test strength of wood structures. The variation in strength which is observed among the type groups is not obviously explainable by the measured properties of the lumber or thickness of plates. Since most failures are plate-related, it might be conjectured that plate details beyond those studied here are responsible for these differences. A principal objective of the present study was to appraise design techniques in current use and these ultimate load tests confirm the conclusion that these commercial designs performed very well with respect to strength when fabricated with good to excellent lumber.

III. LONG-TERM TESTS

TEST FACILITIES

The long-term tests were conducted in a 25 ft x 50 ft unheated, concrete floor slab, wood-frame building located in Brookston, Indiana, 12 miles north of Purdue University. Solid concrete blocks, bricks, and lead weights were used to provide uniform loads. Deflections were measured with apparatus similar to that used for the short-term tests.

TEST UNITS AND MATERIALS

At the beginning of this project, plans were made to test five sets of two each of the parallel-chord floor trusses; four of these sets were to represent types 1 through 4 with specimens randomly selected from the trusses remaining after the short-term test specimens had been chosen. The fifth and the subsequently added sixth sets were designed with an off-center opening. However, off-center floor truss delivery was too late for inclusion in the long-term tests. A standard floor joist setup was substituted for comparison purposes and completed the array of structures included in the long-term experiment.

The two 2 x 10 pieces for the joist setup were chosen from a group of six No. 2 KD Southern Pine 2 x 10s on the basis of their measured modulus of elasticity (MOE). The NDS tabular value of MOE for this grade and species is 1.6 million psi. From edgewise centerpoint load tests on a 16-foot span, the two pieces selected were very close to the tabular MOE. The joist setup involved a 16-foot 5-inch maximum allowable clear span spaced 16 inches on center, supported on 2 x 4 plates and sheathed with ⅝ inch tongue-and-groove underlayment-grade plywood. This sheathing was 4 feet in width and cut into 3-foot lengths with the face grain perpendicular to the axis of the joists. The sheathing was attached with 6d ring-shank nails placed 8 inches on center. While the American Plywood Association (APA) "Plywood Residential Construction Guide" (1) specifies 8d deformed-shank nails for plywood subflooring, the specified nails could not be located among local building materials suppliers.

The APA Construction Guide specifies ⅞-inch-thick tongue-and-groove underlayment-grade plywood for 24-inch-on-center application as with the trusses. After a concerted effort was made to locate the ⅞-inch tongue-and-groove underlayment plywood it was found to be unavailable in the midwest area. Therefore, the decision was made to use ¾-inch-thick tongue-and-groove underlayment on the four sets of floor trusses. The ¾-inch underlayment plywood was cut 3 ft x 4 ft and placed in the same way as for the joist setup and nailed 10 inches on center with 6d ring shank nails.

The truss and joist setups were supported on 2 x 4 reaction units simulating wall bearing on the bottom chords. Wood bridging (1 x 4) was used at the span center on the joist unit only. The floor truss setups had no bridging.

The four pairs of trusses were actually assembled with a 20-inch-on-center spacing rather than the design 24-inch-on-center spacing. This was done in the interest of safety in making test readings within the limited space available for the experiment. However, all loading applied to the test units was based on 24-inch spacing and all data interpretation is made on this basis. Test specimen details are given in Table 3.1.

LOADING

One pair of each of the original four truss groups examined by short-term testing was subjected to long-term testing. Each truss and joist was instrumented with a braided nylon line, scale, and mirror for recording span center deflections as shown in Figure 2.1 but with only gage 2 used.

Each truss pair and the joist pair was flexed twice by preliminary loading to design dead plus full design live prior to initiation of the long-term tests. The dead load included the measured dead weight of the test units. The remainder of dead load and live load was applied by using solid concrete blocks trimmed with bricks and lead weights, placed so as to distribute the load as evenly as possible but also to prevent bridging of the load units. During the preliminary loading, the design load (DL + LL) was carried by the truss and joist pairs for a period of 30 minutes before removing the load.

After pre-loading, the long-term testing began by loading the test units with dead load (DL) plus ½ live load (LL). Here again the dead load included the measured

Table 3.1. Specifications for the long-term test trusses and joists. Truss types are as shown in Figure 1.1 and materials details are similar to those reported in Table 1.1.

Support element	Span* ft.-in.	Depth in.	Underlayment subfloor	Live load psf	Dead load, psf TC/BC
Truss Type 1	21-1	14	¾" ply	40	10/5
Truss Type 2	17-6	12	¾" ply	40	10/10
Truss Type 3	20-11	12	¾" ply	40	10/5
Truss Type 4	22-8	14	¾" ply	40	10/5
2 x 10 joists	17-0	--	⅝" ply	40	10/0

* This is the overall span of the assembly. The clear span of all assemblies is obtained by subtracting 7 inches from this figure.

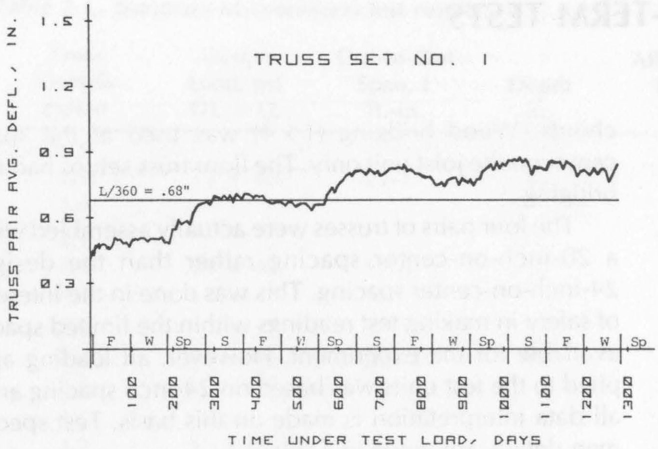

Figure 3.1. Truss type 1.

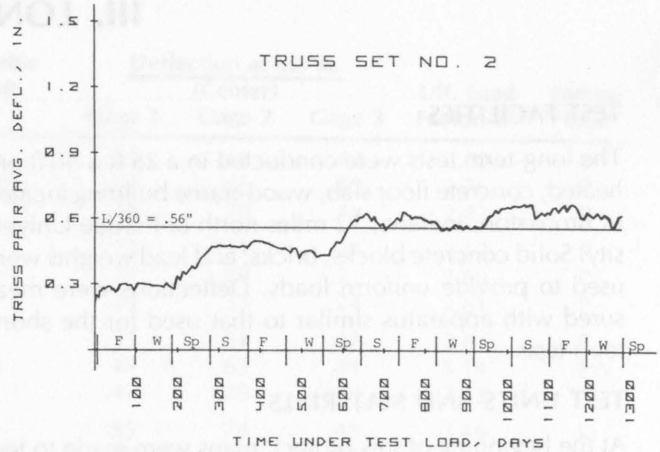

Figure 3.2. Truss type 2.

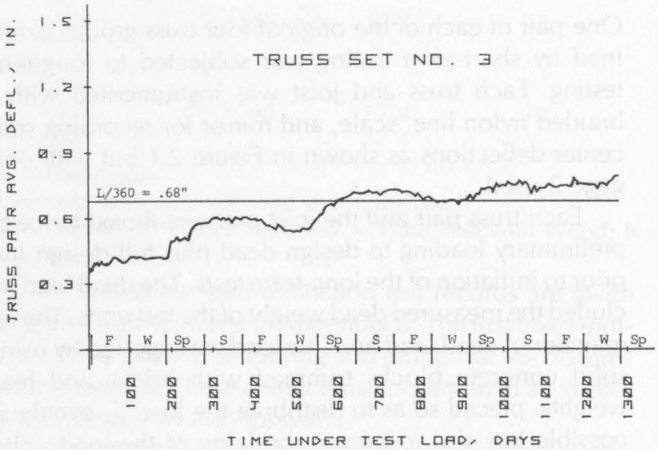

Figure 3.3. Truss type 3.

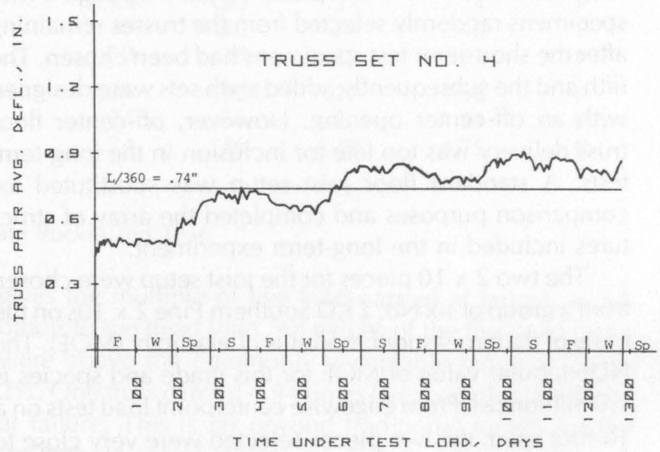

Figure 3.4. Truss type 4.

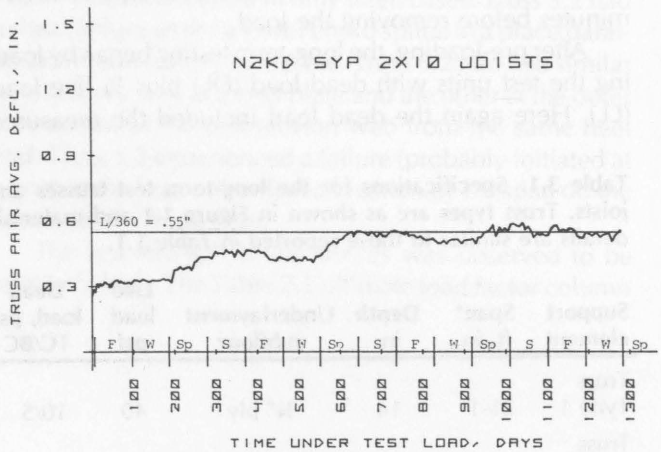

Figure 3.5. No. 2 KD SYP 2 x 10 joists.

Long-term test deflection data for the period beginning September 15, 1977. The test load is 20 pounds per square foot, which is one-half live load. Clear span divided by 360 is shown as a horizontal reference line. While the basic time axis is in days, seasons of the year are also marked with longer vertical line segments.

dead weight of the test units and the remainder of the DL plus ½ LL was applied with solid concrete blocks trimmed to precise values with bricks and lead weights. These loads were also placed so as to avoid bridging of the loading elements. This design dead load plus ½ design live load was considered, by consultation with FHA engineers, to be both adequate and more realistic for long-term testing.

Selected plated joints on the truss units were spray painted to enable detection of possible visible signs of slip during the long term testing if they should occur.

A continuous reading hygrothermograph recording temperature and relative humidity was used to monitor surrounding conditions as well as a standard thermometer both inside and outside the testing building.

Readings were recorded to develop deflection history data for each test unit and are shown plotted in Figures 3.1 through 3.5. The data shown in these figures represent average readings for each pair of test elements. The deflection readings were closely spaced in time for the first days and then were taken at longer intervals according to the observed rate of deflection change. Numerical

Table 3.2. Average centerline deflection of long-term truss and joist setups under dead load plus one-half live load.

Specimen	Initial Deflection	180 days	368 days	550 days	788 days	914 days	1096 days	1278 days	Maximum Deflection
	Centerline Deflection in inches Average of Test Specimen pairs								
Truss Set No. 1 L/360 = .68	.341	.486	.681	.661	.801	.762	.825	.843	.876
Truss Set No. 2 L/360 = .56	.175	.289	.467	.433	.608	.559	.634	.622	.671
Truss Set No. 3 L/360 = .68	.287	.415	.594	.593	.701	.683	.736	.793	.793
Truss Set No. 4 L/360 = .74	.337	.482	.693	.677	.825	.780	.850	.864	.911
Joist Set L/360 = .55	.234	.329	.441	.439	.524	.514	.533	.559	.593

values of average deflections of test element pairs at selected time points are given in Table 3.2.

CONCLUSIONS

All tests units carried their loads for the 1278-day test period without signs of distress of any sort. The spray painted plate connections showed no visible signs of movement within the joints. Strength of these designs seems quite well assured insofar as the performance of these good to better test trusses can be judged under the selected loading.

The deflection records of three and one-half years show patterns that have been observed for bending loads on wood components in previous but unpublished long-term load test experiments. The floor joist set, Figure 3.5, exhibits the creep behavior of lumber. Each spring there is a period of creep followed by some, but not complete, recovery in the following fall and winter. This is clearly an environmental influence associated with gross effects of the seasons. Obvious and simple relationships with the temperature and humidity records are not apparent by a straightforward examination of the data. The joists and three of the truss sets, Figures 3.2, 3.3, 3.5, exhibited deflection creep beyond L/360 in the second year with the trusses showing slightly more creep. The latter effect could be due to non-recoverable (but not observable) deformation in the joints. Truss type 1, Figure 3.1, crept past L/360 in the first year but this is not interpreted as unusual since this was the most highly stressed design.

An over-all appraisal of the deflection behavior from this long-term portion of the study, the deflection data from the short-term tests, and the analytic investigation of deflection in a later chapter, focuses attention on the importance of variation in modulus of elasticity in the chord lumber. The trusses used in these studies were good specimens with good to better chord lumber. Considering the MOE variation possible in visually graded lumber, these experiments suggest that deflection problems can surface from time to time even though designs and fabrication are within specification.

IV. METAL-PLATE JOINT PERFORMANCE

While metal plates have been involved in many test-oriented research programs in the past 20 years and their reliability as structural connectors is now well established, some further investigations were considered to be desirable as a part of this program. The plate study reported in this chapter was presented at a metal-plate wood truss symposium held in St. Louis in 1979 and forms a part of the proceedings from that meeting (12).

This study reports the results of 322 tensile tests of 3" x 3" metal plate connections splicing 2 x 4 lumber. The test specimen (Figure 4.1) and procedure are identical to those given by the Truss Plate Institute (14) for plate-to-wood connector qualification. All plates were 20 gage, of the same manufacture, and all were placed in a zero degree orientation with the long axis of the specimen. The first objectives of study were to observe the amount of variation to be expected from metal plate connections in terms of both their ultimate strength and stiffness as measured by load at a standard amount of slip. Further objectives were to observe the influence of specific gravity, moisture content, and species on the strength and stiffness variables. This is probably the first time for a study of this kind and the results are more exploratory in nature rather than being statistically based on random sampling. The design of subsequent more comprehensive samplings can, however, take advantage of the findings of this study.

LUMBER SAMPLES

The samples were obtained as opportunity permitted. The Southern Pine portion is from five sources. Twenty-four specimens were made from chord stock from previously tested parallel-chord trusses. This lumber was grade marked as No. 1 and No. 2 Dense and was observed to be of excellent quality for these grades. Thirty-five specimens were made from chord stock from trusses load tested in a study of the influence of heel wedges in triangular-shaped trusses. This lumber was all grade marked as No. 1 Dense Kiln Dried 2250f and was also of excellent quality. Twenty specimens were made from No. 2 Kiln Dried stock saved as short pieces from a variety of uses in experimental work at the University of Illinois Small Homes Council laboratory. This lumber was of lower quality than the previously described groups. Twenty specimens were made from short trim ends taken from the manufacturing line at a truss fabrication plant in Indiana. This material was quite variable, as might be expected, and also contained the only significant range of moisture content variation in the entire study. The above 99 specimens were supplemented by 142 Southern Pine samples from a random sample of truss chord 2 x 4 lumber obtained from truss fabricators in Illinois. The total set of test information from these 241 specimens makes up the "Southern Pine" data set as identified in subsequent discussion in this report.

Two sample sets of 30 specimens each of 2100f and 1650f grades of Spruce-Pine-Fir (SPF) represent Machine Stress Rated (MSR) lumber in this study. Full length 2 x 4 pieces were randomly sampled from a mill in northern Alberta, Canada.

The "Illinois Sample" contained 21 pieces of lumber that did not come from Southern Pine species group. These were all visually graded material from Canada or the western United States. Included were No. 1 and Construction grade S-Dry Douglas Fir, No. 1 S-Grn Douglas Fir, Construction and Standard grade S-Dry Hem Fir and No. 1, No. 2, Construction and Standard grade S-Dry Spruce-Pine-Fir.

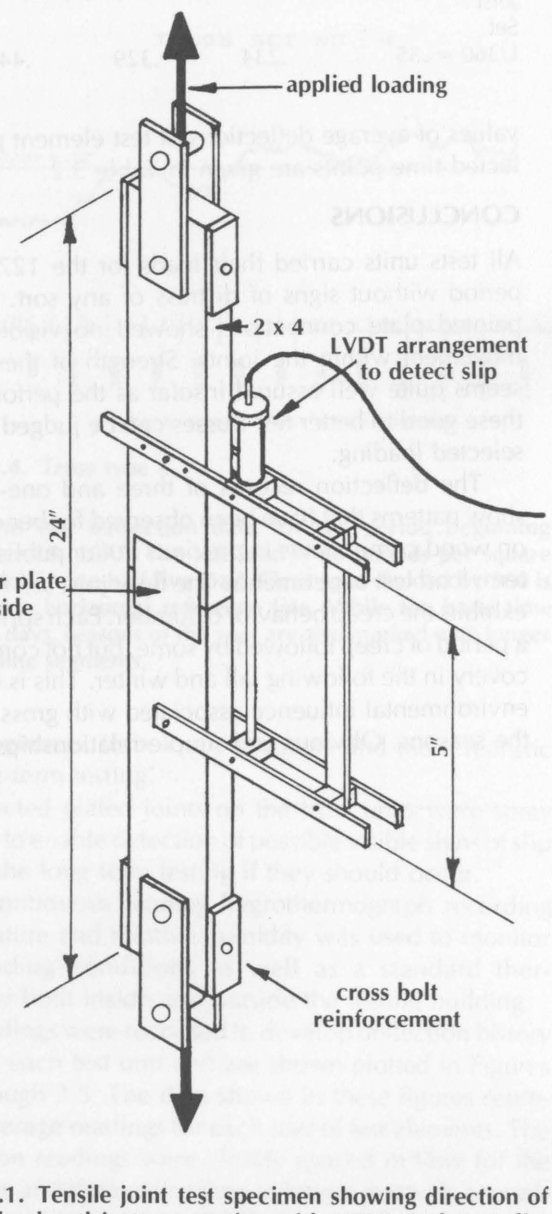

Figure 4.1. Tensile joint test specimen showing direction of applied load and instrumentation with LVDT to detect slip.

SPECIMEN PREPARATION AND TESTING

Specimens were made up by joining two pieces of 2 x 4, one foot long with 3" x 3" 20-gage metal plates having 11/32" long teeth at a density of 8 teeth per square inch. The specimen and its arrangement for test is shown in Figure 4.1. A linear variable differential transformer (LVDT) was used, in conjunction with a calibrated extensometer apparatus, to measure slip of the connection to the nearest .001 inches. Voltage from this instrument, along with an electric reading from the universal testing machine, were fed to a computer interface enabling automated data acquisition. Both the LVDT and the testing machine electric outputs were put through an independent calibration procedure at frequent intervals to assure the quality of data recorded within each interval.

Load was applied to the specimen using a testing machine platen speed of .01 inches per minute, which provided for failure in the desired time range of 5 to 20 minutes. Readings of load and deformation were taken at either 1-second or 10-second time intervals at the option of the operator. The faster rate was used when finer detail of the load-deformation curve was felt to be required. Figure 4.2 shows a typical load-deformation record for one test. The load at .030" total joint slip (.015" per connection area) and the ultimate load were automatically determined by the computer.

The predominant and intended mode of failure was peeling of the plate, which is the normal way in which a steel-to-wood failure occurs. On some occasions (20 out of the 322 cases) the load reached a level that caused tensile failure in the plate itself which is simply described as tearing of the metal in, or very near, the plane of the cut faces of the wood joint. The load at which this occurred was recorded as the ultimate, the same as with the case of plate peeling, which introduces some bias in the results on the high end of the ultimate load distributions.

Table 4.1. Basic statistics from 322 tensile plate-connection load tests.

Sample	Statistics[1]	Ult. Load	Load at .03" Slip	Moisture Content	Specific Gravity[2]
Southern	$\bar{x}$	5101	4569	10.1	.458
Pine	V	.136	.123	.349	.130
	N = 241				
MSR	$\bar{x}$	4871	4223	9.1	.443
2100f	V	.114	.109	.034	.072
SPF	N = 30				
MSR	$\bar{x}$	4328	3880	9.1	.401
1650f	V	.098	.088	.031	.073
	N = 30				
Illinois	$\bar{x}$	4667	4196	9.6	.403
Sample	V	.198	.175	.066	.116
Not So. Pine	N = 21				

[1] $\bar{x}$ = Average; V = Coefficient of Variation; N = Sample Size
[2] Oven-dry weight, green volume basis

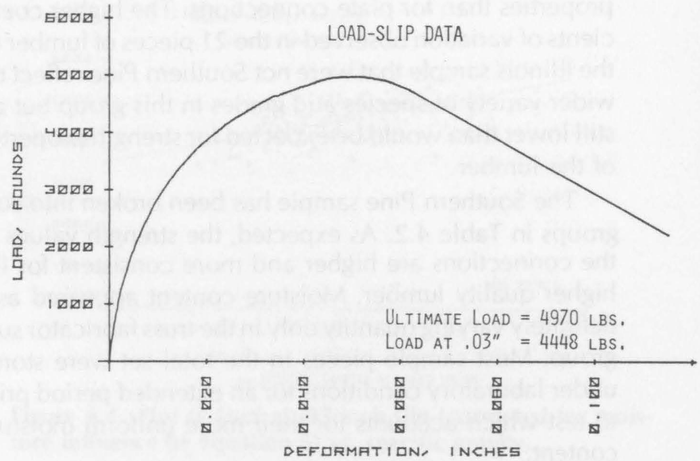

Figure 4.2. Typical joint test load-deflection curve.

Small wood blocks were cut from the 2 x 4 pieces close to the plate at the termination of the test to determine moisture content and specific gravity. When the failure occurred due to plate peeling, the test was cut from the side of the joint in which this failure occurred. Moisture content was determined by the oven drying method. Specific gravity was determined in accordance with ASTM D 2395-69 (2) and adjusted to a green volume basis.

The data obtained from each tensile test is identified and summarized as follows:

P_u - Ultimate load carried by the connection with failure predominantly by plate peeling and otherwise by plate tearing.

P_s - Load at .03" total slip of the connection which is equivalent to .015" slip at each plate-to-wood connection.

M - Moisture content at time of test, o.d. weight determination.

G - Specific gravity of test wood as determined by o.d. weight and volume converted to a green basis.

S - A gross species grouping variable used in a multiple regression analysis having the value 1 for Southern Pine or 0 for other species.

ANALYSIS

The basic statistics are given in Table 4.1 in which the 322 test specimens have been broken into their main groups. A significant feature of these results is the relatively low coefficients of variation obtained for both the ultimate load, P_u, and the load at .03" slip, P_s. These are well below the values obtained for the commonly used structural design properties of lumber and are compatible with the observation from truss testing that plate connection behavior is more consistent from test to test than is lumber strength. This could be an important consideration in the evaluation of probabilistic methods for truss design with more emphasis being needed for variation in lumber properties than for plate connections. The higher coefficients of variation observed in the 21 pieces of lumber for the Illinois sample that were not Southern Pine reflect the wider variety of species and grades in this group but are still lower than would be expected for strength properties of the lumber.

The Southern Pine sample has been broken into subgroups in Table 4.2. As expected, the strength values of the connections are higher and more consistent for the higher quality lumber. Moisture content appeared as a definitely varying quantity only in the truss fabricator subgroup. Most sample pieces in the total set were stored under laboratory conditions for an extended period prior to test which accounts for their more uniform moisture content.

Specific gravity, G, which will be discussed more completely below, is an important predictor variable relating to the strength values P_u and P_s. The relationship can be generally observed by scanning the averages in Tables 4.1 and 4.2. Another aspect of specific gravity can be noted in these tables which relates to the grading processes. The National Design Specification (NDS) (8) gives, in Table 8.1A, what are taken to be average specific gravity values (converted to green volume basis) of .38 and .48 for the species groups of Spruce-Pine-Fir and Southern Pine, respectively. The two machine grades of SPF reported here had average specific gravities of .401 and .443, which are both above the .38 value and are arranged in relation to their grades. This sorting by grading machines into ascending specific gravity groups according to grade and with the more common grade of 1650f falling above published values, such as .38, has been previously observed in unpublished industrial quality control records. The visually graded Southern Pine (Table 4.2) also shows a scaling of average specific gravity by grade but the published value of .48 relates more to the data concerning the dense grades.

Multiple regression analyses were performed using an SPSS (9) statistical package available in the library of the Purdue University Computing Center. The first runs, using all of the data in Table 4.1, were linear regressions using ultimate load, P_u, and the slip load, P_s, as the dependent variables against specific gravity, G, moisture content, M, and species group, S. These regressions were performed using a stepwise method which sorted out the most influential independent variables and included them in descending order which turned out to be G, M, and S. All coefficients were significant well beyond the one percent level, indicating that the species group variable, S, was important and that the data should be sorted by species for further useful analyses.

The 241 Southern Pine cases were subjected to the same type of analyses resulting in the following equations:

$$P_u = 6109G - 44.46M + 2751 \quad (1)$$

with: mult. corr. coef. R = .605

simple corr. P_u vs. G; r = .563

simple corr. P_u vs. M; r = − .317

$$P_s = 5093G - 47.19M + 2712 \quad (2)$$

with: mult. R = .657

P_s vs. G; r = .589

P_s vs. M; r = − .389

Equations (1) and (2) are predictor equations indicating the general nature of the relationships among the variables. The variation in moisture content among the samples was generally quite small and could have forced the variable M into a much more minor role than it would have if more fully distributed.

A two-dimensional picture of the separate influences of specific gravity and moisture content on the variables P_u and P_s can be gained by utilizing equations (1) and (2) to develop corrections for moisture content and for specific gravity. The correction is of the form

$$P' = P - (x - \bar{x})b \quad (3)$$

where x is the influencing variable to be eliminated, such as M or G

$\bar{x}$ is the mean of the influencing variable

b is the coefficient of the influencing variable from equation (1) or (2).

As an example, P_u corrected for moisture content in the 241-specimen Southern Pine sample yields P'_u values adjusted for moisture content deviations from their average value of 10.09 percent.

$$P'_u = P_u - (M - 10.1)(-44.5) \quad (4)$$

Table 4.2. Breakdown of 241-piece Southern pine sample.

Sample	Statistics[1]	Ult. Load	Load at .03" Slip	Moisture Content	Specific Gravity[2]
Parallel chord	$\bar{x}$	5614	4993	7.7	.486
truss	V	.116	.097	.090	.133
N1D, N2D	N = 24				
Slope chord	$\bar{x}$	5502	4886	9.4	.483
truss	V	.098	.092	.073	.072
N1DKD	N = 35				
Small Homes	$\bar{x}$	5085	4517	9.5	.430
Council	V	.110	.091	.080	.101
N2	N = 20				
Truss fabricator assorted	$\bar{x}$	4430	3884	19.0	.424
	V	.157	.125	.390	.113
grades	N = 20				
Illinois	$\bar{x}$	5013	4524	9.5	.456
Sample	V	.129	.116	.092	.138
So. Pine	N = 142				

[1] $\bar{x}$ = Average; V = Coefficient of Variation; N = Sample Size
[2] Oven-dry weight, green volume basis

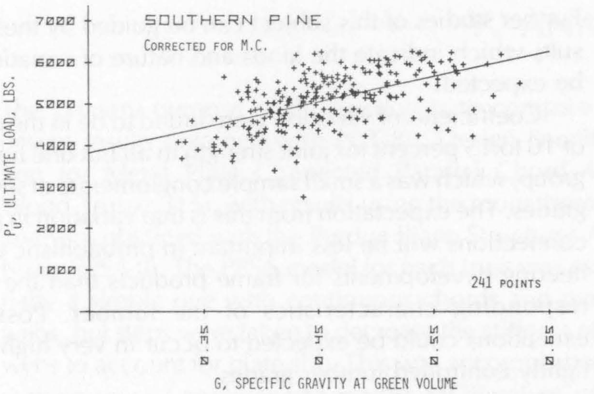

Figure 4.3. Plot of ultimate joint strength (corrected for moisture influence by Equation 4) vs. specific gravity.

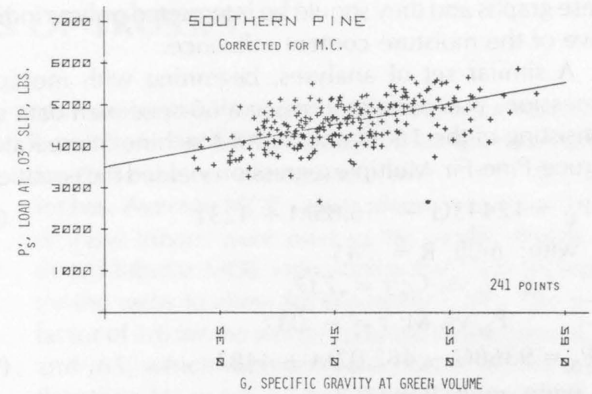

Figure 4.4. Plot of load at .03-inch slip (corrected for moisture influence by Equation 5) vs. specific gravity.

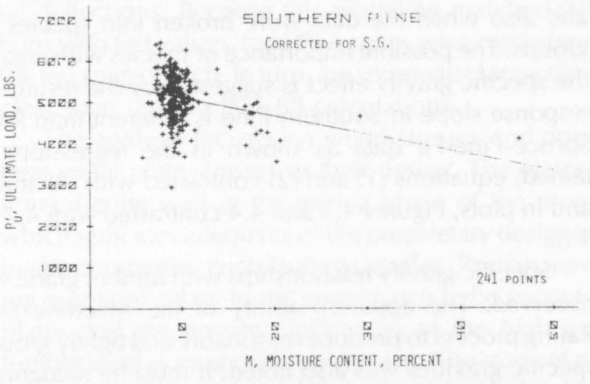

Figure 4.5. Plot of ultimate joint strength (corrected for specific gravity influence by Equation 6) vs. moisture content.

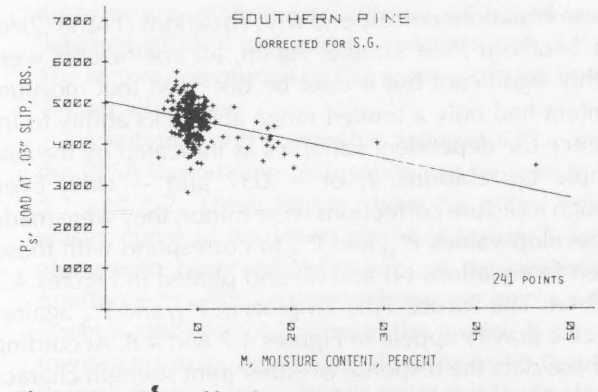

Figure 4.6. Plot of load at .03-inch slip (corrected for specific gravity influence by Equation 7) vs. moisture content.

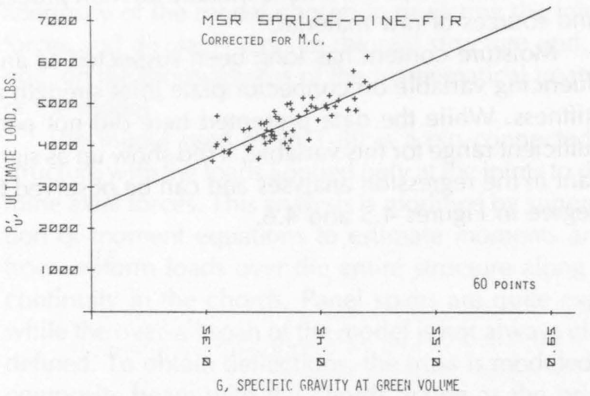

Figure 4.7. Plot of ultimate joint strength (corrected for moisture influence by Equation 8) vs. specific gravity.

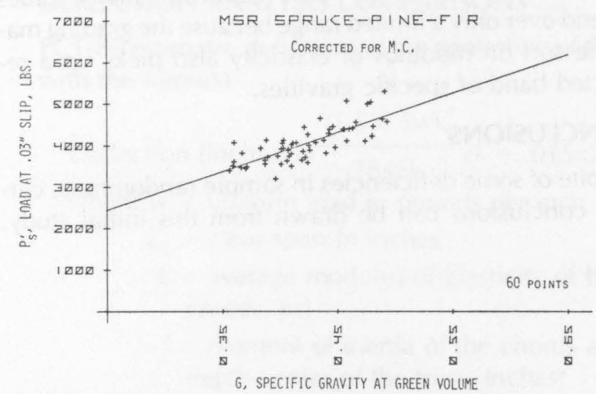

Figure 4.8. Plot of load at .03-inch slip (corrected for moisture influence by Equation 9) vs. specific gravity.

When the P'_u values are plotted against specific gravity, the plot (Figure 4.3) shows the influence of specific gravity on the ultimate strength of the connections with adjustments having been made on the ultimate strength for the influence of moisture content above or below its average value. The same process yields a similar set of values, P'_s for the slip load for the 241-specimen southern Pine sample.

$$P'_s = P_s - (M - 10.1)(-47.2) \tag{5}$$

These values plotted against specific gravity appear in Figure 4.4.

Equation (3) can also be used in a similar way to adjust the data for specific gravity.

$$P'_u = P_u - (G - .458)(6109) \tag{6}$$

$$P'_s = P_s - (G - .458)(5093) \tag{7}$$

The plots of P'_u and P'_s against moisture content, M, appear in Figures 4.5 and 4.6 respectively. The lack of range distribution in moisture content is clearly seen in these graphs and they should be interpreted only as indicative of the moisture content influence.

A similar set of analyses, beginning with multiple regression, was performed using a 60-specimen data set consisting of the 1650f and 2100f Machine Stress Rated Spruce-Pine-Fir. Multiple regression yielded the equations

$$P_u = 12443G - 536.85M + 4231 \quad (8)$$

with: mult. R = .745
P_u vs. G; r = .737
P_u vs. M; r = − .037

$$P_s = 9368G - 482.02M + 4482 \quad (9)$$

with: mult. R = .767
P_s vs. G; r = .700
P_s vs. M; r = − .089

These equations correspond with equations (1) and (2) for the Southern Pine sample. Again, all coefficients were highly significant but it must be observed that moisture content had only a limited range and lacks ability to influence the dependent variables as indicated by the low simple correlations, r, of − .037 and − .089. Even though moisture corrections were minor, they were made to develop values P'_u and P'_s to correspond with those given in equations (4) and (5) and plotted in Figures 4.3 and 4.4. The Spruce-Pine-Fir plots of P'_u and P'_s against specific gravity appear in Figures 4.7 and 4.8. According to these data the response of either joint strength characteristic to increase in specific gravity appear to be steeper for Spruce-Pine-Fir than for Southern Pine. It should also be noted that the Spruce-Pine-Fir specific gravity values extend over only a limited range because the grading machine sort on modulus of elasticity also picks up a restricted band of specific gravities.

CONCLUSIONS

In spite of some deficiencies in sample randomness, certain conclusions can be drawn from this initial study. Further studies of this subject can be guided by these results which indicate the kinds and nature of variation to be expected.

Coefficients of variation were found to be in the order of 10 to 15 percent for joint strength in all but one lumber group, which was a small sample conglomerate of several grades. The expectation from this is that variation in plate connections will be less important in probabilistic engineering developments for frame products than the corresponding characteristics of the lumber. Possible exceptions could be expected to occur in very high and tightly controlled lumber grades.

Specific gravity of the lumber is definitely related to the strength and stiffness of the plate connections. This characteristic was apparent in analyses of all of the data and also when the data were broken into species subgroups. The possible importance of species with regard to the specific gravity effect is suggested by the results. The response slope in Southern Pine is different than for the Spruce-Pine-Fir data as shown in the regressions obtained, equations (1) and (2) contrasted with (8) and (9), and in plots, Figures 4.3 and 4.4 contrasted with 4.7 and 4.8.

Specific gravity relationships with lumber grade were observed. The apparent ability of the Machine Stress Rating process to produce reasonably and tightly grouped specific gravities was also noted. It must be recognized, however, that only one machine in one mill using lumber from a particular region was involved. Generalization cannot be made at this time to include all MSR processes and sources of raw material.

Moisture content has long been suspected as an influencing variable on connector plate joint strength and stiffness. While the data presented here did not permit sufficient range for this variable, it did show up as significant in the regression analyses and can be observed to a degree in Figures 4.5 and 4.6.

V. ANALYSIS OF TRUSSES

The primary purpose of this section is to compare the design results obtained with PCT-80, Design Specification for Metal Plate Connected Parallel Chord 4 x 2 Wood Trusses (15), with results using the more thorough analyses obtained with the Purdue Plane Structures Analyzer (PPSA II). The PPSA model for each truss was essentially a simple one with continuous chords and pinned webs, but steps were taken to decrease the stiffness of the webs to account for plate slip. This was accomplished by making several analyses varying the MOE values of the webs and choosing that value in each case that yielded close agreement between the theoretical and experimental deflections. Because this modeling matched deflections with test values, the PPSA spans were made equal to the test spans which, in turn, are somewhat larger than the clear span used in PCT-80 calculations.

The analysis focuses on wood stresses and does not treat metal plate stresses or their design. The plates performed quite well in the testing phase of the program, which indicates adequacy of the proprietary designs used by the companies contributing samples. Primary remaining questions relate to the consistency to be expected of plates and the possible influences of the high quality lumber used in most of the specimens. The study of plates reported in the previous chapter furnishes helpful information covering these questions.

Precision in structural analysis depends, first, on the adequacy of the model chosen in depicting the internal forces and displacements in the real structure and, second, on the thoroughness of the mathematical treatment of the model.

The model used by PCT-80 is a pin-connected line structure with the loads applied only at the joints to determine axial forces. This analysis is modified by superposition of moment equations to estimate moments arising from uniform loads over the entire structure along with continuity in the chords. Panel spans are quite explicit while the over-all span of the model is not always clearly defined. To obtain deflections, the truss is modeled as a composite beam with the chords acting as the primary stiffness elements. No web or other shear function is included directly in this deflection model except for a 1.33 multiplier in the deflection formula to account for shear deformation of the structure as a whole. An empirical factor is used to modify the deflection calculation where the rectangular opening is not centered in the span.

PPSA II is a system which permits modeling in an infinite variety of ways and produces a virtual work solution of the model that includes forces, moments, and shears at all points in the structure, along with a complete description of displacements. The PPSA II model chosen for the 4 x 2 truss consists of continuous chords and pin connected web members. This model was tuned to the short-term design live-load deflection values for trusses 1.1, 1.2 and 1.3, in the following way. The observed deflections for these trusses were, respectively, .67, .86, and .70 inches. Average MOE values observed within the chords of these trusses were used in the model chords, but reduced tabular MOE values from the NDS (8) were used for the webs to allow for slip at the joints. The reduction factor of 1/6 for the webs produced deflections of .67, .75 and .65, which represents a closer fit to the deflection data than 1/7 or 1/5. Factors consisting of whole fractions represent as fine a distinction among factor values as was considered practical. The other truss types were treated the same way as shown in Table 5.1 with good agreement between PPSA II deflections and observed deflections. The factors determined by this process ranged from 1/6 to 1/4 for all trusses.

Additional and somewhat independent support for the PPSA II modeling philosophy can be seen in Figures 5.1 and 5.2. These figures show the entire theoretical elastic curve of the lower chord of trusses 5.2 and 5.3 along with observed deflections at the centerline and quarter points. The agreement between theory and experiment is strongly supportive of the model. It can also be mentioned from experimental observation that the deflected shape of truss chords under test loads closely resembles the theoretical deflection curve.

DEFLECTION ANALYSIS COMPARISONS

PCT-80 estimates deflection at the centerline of the span with the formula

$$\text{Deflection (inches)} = \frac{(1.33)\,5wL_S^4}{384EI}(1 + .015x)$$

where w = uniform load in pounds per inch

L_S = clear span in inches

E = average modulus of elasticity of both chords, psi

I = moment of inertia of the chords about depth center of the truss, inches^4

x = offset distance from center of rectangular opening to center of truss span, inches, and not to exceed 15 inches

Application of this formula to the six truss patterns tested results in the deflections shown in the PCT-80 column of Table 5.1. It must be particularly noted that the measured E-values for the chords of each individual truss were used in the deflection formula. The results obtained with all truss types are entirely satisfactory and generally as good as those obtained with PPSA II. The trusses with off-center openings, types 5 and 6, each had offset distance values of 20.25 inches which is beyond the limit and yet the PCT-80 deflection formula still performed reasonably

Table 5.1. **Calculated deflections by PCT-80 and PPSA II are shown along with observed values. Table values of E for the web members were reduced in the PPSA II analyses by the fractional amount shown in the right-hand column. Measured values of E in the chords were used in both PCT-80 and PPSA II deflection calculations.**

Truss Ident.	Overall Span ft.-in.	Depth in.	PCT-80 Deflection in.	PPSA II Deflection in.	Observed Deflection in.	For PPSA II Web E/Table E
1.1			.66	.67	.67	
1.2	21-1	12	.78	.77	.86	1/6
1.3			.62	.65	.70	
2.1			.30	.31	.31	
2.2	17-6	12	.29	.31	.29	1/4
2.3			.29	.31	.33	
3.1			.44	.46	.51	
3.2	20-11	14	.66	.63	.63	1/6
3.3			.80	.75	.70	
4.1			.70	.75	.74	
4.2	22-8	14	.68	.73	.71	1/5
4.3			.65	.70	.64	
5.1			.35	.36	.35	
5.2	17-4	12	.30	.31	.30	1/5
5.3			.26	.28	.28	
6.1			.42	.39	.38	
6.2	17-4	12	.43	.40	.36	1/4
6.3			.43	.40	.36	

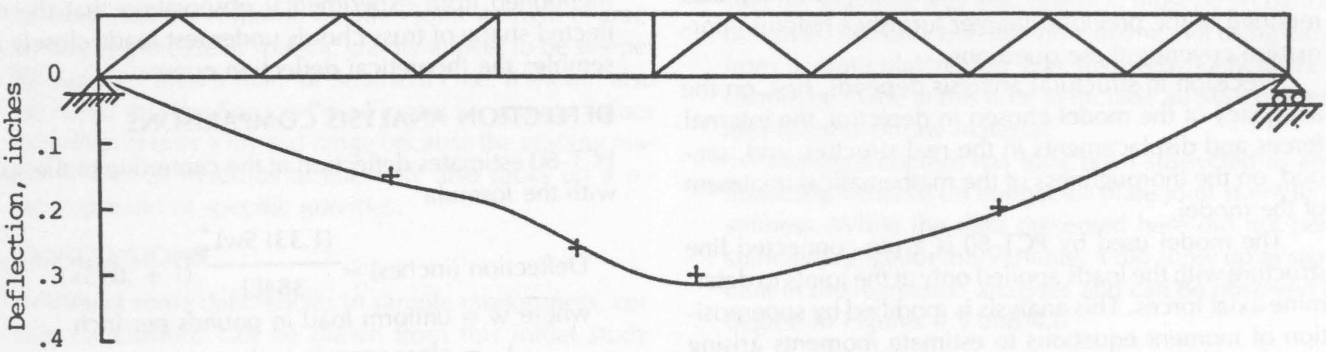

Figure 5.1. Truss 5.2.

Theoretical deflections under uniform live load applied to the upper chord are shown as a continuous line. Experimental deflections are plotted with cross symbols.

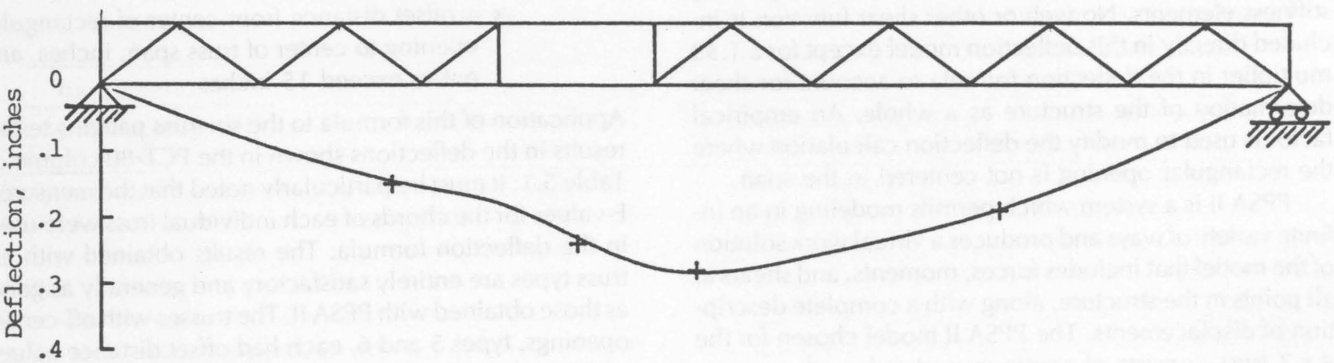

Figure 5.2. Truss 5.3.

well. Such accuracy is obtainable only with an accurate estimate of chord E-values, however, and the deflection performance of other trusses will vary with chord lumber stiffness.

STRENGTH ANALYSIS COMPARISONS

Once the PPSA II truss model was finalized and tuned with deflection comparisons, it was then used for strength analyses involving live plus dead loads. One truss with appropriate E-values for chord and webs was analyzed for each trial. Four types of studies are presented: the first deals with maximum stress conditions in the symmetric trusses; the second with the same subject treatment of offset opening trusses; the third with maximum web stresses; and a fourth separate study treats concentrated load influences.

For the first two studies, the criterion of importance is the combined stress index, CSI, as given in section 3.10 of the 1977 National Design Specification (8). The CSI is the sum of two fractions where the first is the ratio of axial stress to allowable axial stress, and the second is the ratio of bending stress to allowable bending stress. A significant change in the 1977 NDS from previous versions is its treatment of allowable axial stress in the case of flexure plus axial compression. PPSA II incorporates a reasonably precise method for determining equivalent column lengths. It can be generally stated that, in none of the cases covered in this report, did the equivalent column length to depth ratio in the chords ever exceed that for a short column in a highly stressed member. Further, no allowance was attempted for the column reinforcing effect produced by the attached subfloor. These observations should not, however, be immediately extended to 4 x 2 trusses of radically different patterns or designed for multiple or cantilevered supports since these variations were not investigated.

The truss drawings received from the industrial designers varied in the amount of supplementary information and slightly in the calculation of member stresses when these were reported. In order to make an orderly appraisal of PCT-80, uniform policies of analysis were adopted which produced some slight differences in values from those obtained from the designers. Chord forces were calculated on the basis of an eighth moment, total load, the clear span, and the distance between chord centroidal axes. PCT-80 provides for the use of any suitable engineering method of analysis provided it accounts for bending moment due to loads acting on chords between panel points, bending moment induced into the chords by the over-all deformation of the truss, and bending moment produced at a rectangular opening due to transfer of shear across the opening. A simplified method of analysis is also given in PCT-80 that may be used under a given set of constraints as to loads, rectangular openings, and symmetry. The trusses analyzed here meet the constraints except for excess in the allowable 15-inch offset of the rectangular openings in trusses types 5 and 6, which were offset 20.25 inches. The simplified method of PCT-80 was used in all analyses reported in the tables and was applied to all trusses even though types 5 and 6 do not quite qualify for this method.

Table 5.2 gives a comparative breakdown of maximum CSI values in both chords as calculated by PCT-80 and PPSA II. In each case, a specific truss was used with its measured E-values for the more complete PPSA analysis. The type 1 truss as represented by 1.1 was somewhat extended in design by either method of analysis. It happened that this truss was also most frequently used in other investigations because of its popular web configuration and greater span-depth ratio. Other CSI calculations seen later in this report must be evaluated in terms of the PPSA II CSI value of 1.123 as obtained in this benchmark situation.

A comparison of CSI values for the two systems of analysis must recognize the fact that the analytical spans are different. The use of clear span with PCT-80 yields the lowest CSI values to be expected in practice while the PPSA II spans are based on experience in more detailed modeling and confirmation experiments with trusses and related frames. Chord stresses as calculated by PCT-80

Table 5.2. Strength analysis of the symmetric trusses. All trusses have 40 psf live plus 10 psf dead loads on the upper chords. Truss 2.1 has 10 psf dead load on the lower chord; all other trusses have 5 psf.

	PCT-80[1]						PPSA II[2]					
	Upper Chord			Lower Chord			Upper Chord			Lower Chord		
Truss Ident.	Force lbs	Moment lb-in.	CSI	Force lbs	Moment lb-in.	CSI	Force lbs	Moment lb-in.	CSI	Force lbs	Moment lb-in.	CSI
1.1	6604	391	.862	6604	250	1.084	6682	537	.916	6682	336	1.123
2.1	4906	402	.842	4906	220	.976	5095	576	.933	5095	262	1.022
3.2	5457	323	.890	5457	179	1.057	5540	449	.948	5540	206	1.082
4.1	6437	392	.843	6437	212	1.047	6520	522	.893	6520	276	1.079

[1] The PCT-80 analyses were made on the basis of clear span (7 inches shorter than the overall span).
[2] The PPSA II analyses were made on the basis of a span 4 inches shorter than the overall span for all trusses except 2.1. Truss 2.1 has a single vertical web at each end and span was defined for this truss by the centerlines of these webs (1½ inches shorter than the overall span).

Table 5.3. Strength analysis of the off-center-opening trusses. All trusses have 40 psf live plus 10 psf dead load on the upper chord and 5 psf dead load on the lower chord.

	PCT-80[1]						PPSA II[2]					
	Upper Chord			Lower Chord			Upper Chord			Lower Chord		
Truss Ident.	Force lbs	Moment lb-in.	CSI	Force lbs	Moment lb-in.	CSI	Force lbs	Moment lb-in.	CSI	Force lbs	Moment lb-in.	CSI
5.2	4230	1588	.968	4230	1336	1.060	4288	1405	.917	4484	1265	1.077
6.2	4230	1416	.914	4230	1267	1.039	4302	1335	.897	4501	1153	1.044
1.1M[3]	6520	900	1.011	6520	906	1.388	6621	848	1.047	6614	1152	1.366
4.1M[3]	6393	802	.966	6393	683	1.187	6486	806	.977	6486	708	1.208

[1] The PCT-80 analyses were made on the basis of clear span (7 inches shorter than the overall span).
[2] The PPSA II analyses were made on the basis of a span 4 inches shorter than the overall span.
[3] These trusses have been modified from their original symmetric design by placing the center of the rectangular opening at one-half the standard panel away from the center of the span.

are, for the most part, proportional to the square of the span and if the CSI values of PCT-80 are multiplied by the ratio of the square of the PPSA II span to the square of the PCT-80 span, new PCT-80 CSI values of 1.111, 1.005, 1.084, and 1.071 result for truss types 1 through 4, respectively. They compare very closely with the corresponding PPSA II values of 1.123, 1.022, 1.082, and 1.079, respectively.

Table 5.3 shows the results of similar calculations with the two analytic methods for the two tested truss types 5 and 6 along with two trusses that were modified forms of truss types 1 and 4. The latter modification was made by offsetting the center rectangular opening by one half standard panel. The CSI results bear a similar relationship between analytic systems to those shown in the symmetric truss cases. Adjustment of the PCT-80 CSI values for trusses 5.2, 6.2, 1.1M, and 4.1M produce figures adjusted by ratio of squares of spans of 1.092, 1.070, 1.422, and 1.214, respectively, which compare with corresponding PPSA II values of 1.077, 1.044, 1.366 and 1.208. This places PCT-80 in a slightly conservative relative position. It should again be noted that the offsets of trusses 5.2 and 6.2 were over the 15-inch limit of PCT-80, but this analytic system still appears to be adequate.

Forces in the first web member are examined in Table 5.4. The precision obtained for these members is considered typical of that for the other web members in the trusses. The primary column designations of "left" and "right" in this table relate to the rectangular opening being placed to the right of center as viewed if the opening is offset at all. The first four truss types are symmetric and only the left web forces are given. For comparison purposes, the clear span was used in calculating the reaction for PCT-80 analyses and the angular position of the web members was taken to be exactly the same as that used in the PPSA II analog. PCT-80 calculations in the table are forces from a pin-connected structure with loads concentrated at the joints. These are seen to be slightly lower than those calculated by PPSA II. A longer analytic span would close the gap between values. The offset trusses, 1.1M and 4.1M, which are within ½ panel of symmetric, show little difference in calculated web forces between the left and right ends.

STRENGTH ANALYSES UNDER CONCENTRATED LOADS

While PCT-80 is specific in Section 202.1 concerning concentrated loads, a truss somewhat underdesigned for uniform loading was investigated to ascertain its stress condition if high but potentially possible concentrated loads were applied. Truss 1.1 was selected for analysis because of its great span-depth ratio, keeping in mind the fact that the basic CSI for this truss is 1.123, Table 5.2. Interpretation of other case analyses of this truss must be made in light of its basic overstress in the benchmark design case. There is no loss in precision in this situation since the short equivalent columns of the upper chord render the entire structure as being linear in terms of the CSI design criteria. Other CSI values from other load cases for this truss can be scaled in proportion to the benchmark case and to each other.

An appraisal of floor loadings by the American Plywood Association showed that water heaters and

Table 5.4. Web member forces in pounds as calculated by PCT-80 and PPSA II. The first four trusses are symmetric. The last four have offset rectangular openings placed to the right of center.

Truss Ident.	Left PCT-80	Left PPSA II	Right PCT-80	Right PPSA II
1.1	1890	1932	--	--
2.1	1484	1546	--	--
3.2	1474	1498	--	--
4.1	2236	2292	--	--
5.2	1456	1496	1456	1505
6.2	1456	1507	1456	1516
1.1M*	1890	1932	1890	1931
4.1M*	2236	2290	2236	2293

* These trusses have been modified from their original symmetric design by placing the center of the rectangular opening at one half the standard panel away from the center of the span.

food freezers could apply the most severe, commonly encountered residential concentrated loads. From this information a concentrated load group was assembled as a "worst condition" consisting of two legs of a full water heater exerting 325 pounds each and two legs of a full food freezer exerting 270 pounds each. The load group, including their spacings and resultant, are shown in Figure 5.3(a).

Dead loads for the analysis were calculated to obtain reasonably realistic values. The assembly of dead load is as follows:

Upper Chord

Floor covering	1.00
½″ particleboard underlayment	1.67
⅝″ plywood sheathing	1.88
½ of truss	1.20
Total	5.75 pounds/square foot

Lower Chord

⅝″ gypsum board	2.60
Pipes, insulation, etc.	1.45
½ of truss	1.20
Total	5.25 pounds/square foot

In addition to the dead load, 10 psf (¼ of the floor live load) was added to the upper chord in panels outside of those influenced by the concentrated loads. In other words, panels falling between the furthest left and the furthest right concentrated load, plus the panels in which these two loads fall were not subjected to the 10 psf live load.

Thirteen cases of load arrangement were subjected to PPSA II analysis. The cases are differentiated from one another by the location of the concentrated loads along the span. These 13 positions are indicated in Figure 5.3(b) by

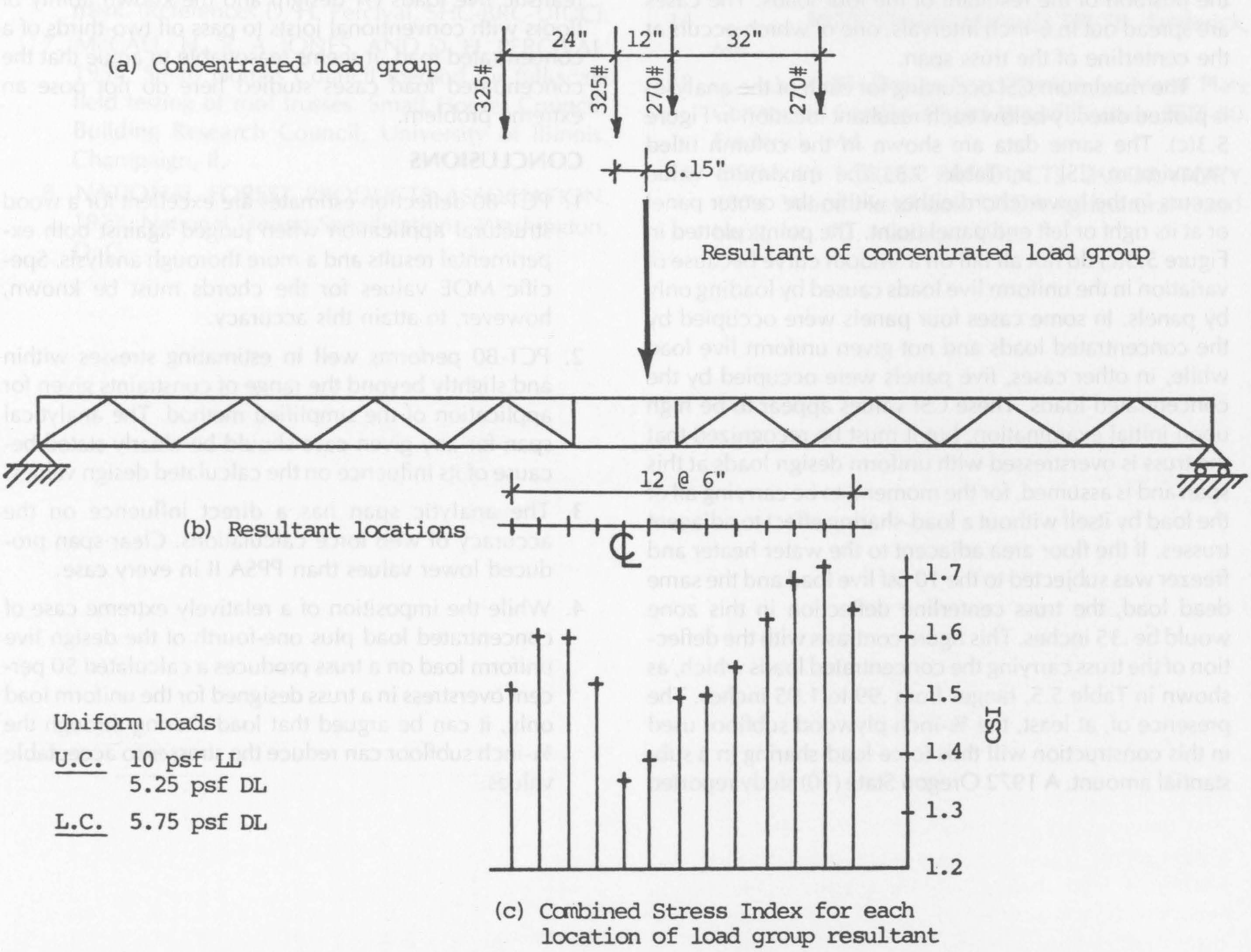

Figure 5.3. Analysis of Truss 1.1 carrying a group of four concentrated loads (a) representing freezer and water heater loads. Spans not occupied by concentrated loads were subjected to the uniform loads indicated. The load group was shifted to 13 different locations as shown by the location of the group resultant (b). Combined stress indices for each of these 13 cases are shown plotted below in (c).

Table 5.5. Summary of concentrated load analysis for Truss 1.1. Loads and their location on the truss are shown in Figure 4.3.

Resultant Location with respect to Span Center	Maximum CSI	Calculated Center Defl., in.	Proportional CSI
24"L	1.51	1.01	1.34
18"L	1.60	1.05	1.42
12"L	1.60	1.04	1.42
6"L	1.52	1.03	1.35
Center	1.35	1.00	1.20
6"R	1.39	1.02	1.24
12"R	1.49	1.04	1.33
18"R	1.50	1.02	1.34
24"R	1.55	1.03	1.38
30"R	1.62	1.03	1.44
36"R	1.70	1.03	1.51
42"R	1.72	1.03	1.53
48"R	1.65	.99	1.47

the position of the resultant of the four loads. The cases are spread out in 6-inch intervals, one of which occurs at the centerline of the truss span.

The maximum CSI occurring for each of the analyses is plotted directly below each resultant location in Figure 5.3(c). The same data are shown in the column titled "Maximum CSI" in Table 5.5. The maximum value occurs in the lower chord either within the center panel or at its right or left end panel point. The points plotted in Figure 5.3(c) do not all fall on a smooth curve because of variation in the uniform live loads caused by loading only by panels. In some cases four panels were occupied by the concentrated loads and not given uniform live load while, in other cases, five panels were occupied by the concentrated loads. These CSI values appear to be high upon initial examination, but it must be recognized that the truss is overstressed with uniform design loads at this span and is assumed, for the moment, to be carrying all of the load by itself without a load-sharing effect to adjacent trusses. If the floor area adjacent to the water heater and freezer was subjected to the 10 psf live load and the same dead load, the truss centerline deflection in this zone would be .35 inches. This figure contrasts with the deflection of the truss carrying the concentrated loads which, as shown in Table 5.5, ranges from .99 to 1.05 inches. The presence of, at least, the ⅝-inch plywood subfloor used in this construction will thus force load-sharing in a substantial amount. A 1972 Oregon State (10) study reported by Polensek, Atherton, Corder and Jenkins involving ½-inch plywood subfloor and conventional joists concluded that two-thirds of a concentrated load on one joist is distributed to six adjacent joists.

A further consideration relates to the overstress in the benchmark truss having a CSI of 1.123. Dividing the maximum CSI values in Table 5.5 by 1.123 produces the column in this same table labeled "Proportional CSI." These values represent combined stress indices that would be obtained from this truss if it were designed so that the uniform loading 40 psf live and 15 psf dead loads produced a CSI of 1.00. Thus, the critical case where the resultant of the concentrated loads lies 42 inches to the right of center represents approximately a 50 percent overstress (CSI = 1.53). A one-third load share to adjacent trusses would bring this index down to within allowable amounts. In view of the relationships between deflections of trusses carrying concentrated loads and realistic live loads (¼ design) and the known ability of floors with conventional joists to pass off two-thirds of a concentrated load, it seems reasonable to argue that the concentrated load cases studied here do not pose an extreme problem.

CONCLUSIONS

1. PCT-80 deflection estimates are excellent for a wood structural application when judged against both experimental results and a more thorough analysis. Specific MOE values for the chords must be known, however, to attain this accuracy.
2. PCT-80 performs well in estimating stresses within and slightly beyond the range of constraints given for application of the simplified method. The analytical span for any given case should be clearly stated because of its influence on the calculated design values.
3. The analytic span has a direct influence on the accuracy of web force calculations. Clear span produced lower values than PPSA II in every case.
4. While the imposition of a relatively extreme case of concentrated load plus one-fourth of the design live uniform load on a truss produces a calculated 50 percent overstress in a truss designed for the uniform load only, it can be argued that load sharing through the ⅝-inch subfloor can reduce the stresses to acceptable values.

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APPENDIX A. MANUFACTURERS TRUSS DRAWINGS

(SPIB 1977)
LUMBER SPECIFICATIONS
TOP CHORD 4x2 (3½x1½)
NO. 1 DENSE K.D. SOUTHERN PINE
BOTTOM CHORD 4x2 (3½x1½)
NO. 1 DENSE K.D. SOUTHERN PINE
WEB MEMBERS 4x2 (3½x1½)
CONSTRUCTION LF. DOUGLAS FIR
OR NO. 3 K.D. SOUTHERN PINE

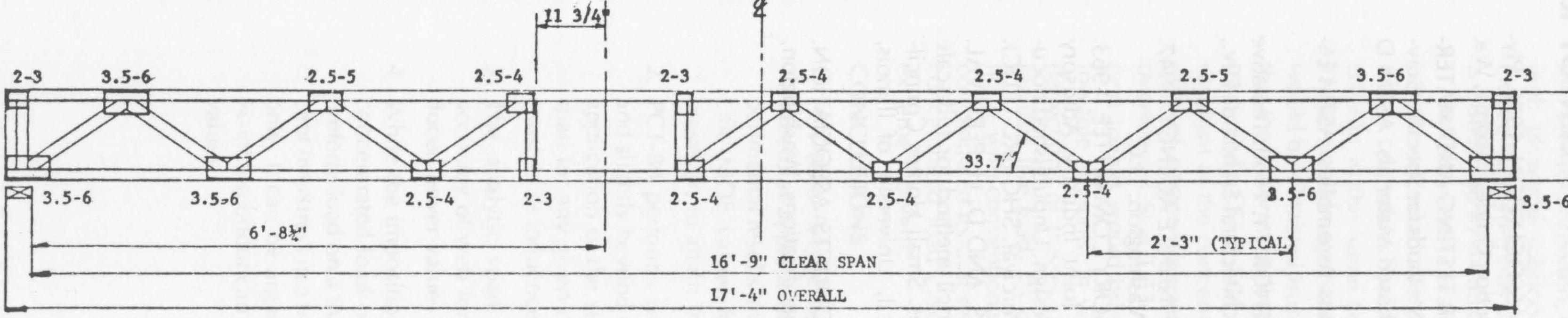
DEFLECTION ANALYSIS SOUTHERN PINE

DEAD LOAD DEFLECTION 0.11"
LIVE LOAD DEFLECTION 0.30"

TOTAL LOAD DEFLECTION 0.42"
ALLOWABLE L.L. DEFL (LC/360.)=0.56"
RECOMMENDED CAMBER 0.30"

NOTE: TRUSS PLATED FOR SOUTHERN PINE

	T/C		B/C		WEB		WEB		VERT
T 1=	0.00	B 1=	1104.53	W 1=-	1399.48			V 1=	0.0
T 2=-	1970.57	B 2=	2836.62	W 2=	1097.31	W 3=-	1097.31		
T 3=-	3384.69	B 3=	3932.75	W 4=	694.42	W 5=-	694.42		
T 4=-	4162.84	B 4=	4392.93	W 6=	291.54	W 7=-	291.54		



M-20
MODEL 20, 20 GA. PLATE WITH
BASIC DESIGN VALUES OF 190
psi [illegible] HEMLOCK AND [illegible]
psi [illegible] DOUGLAS FIR

TrusWal-Ronel plates are formed from 18 and 20 gauge, Grade A, hot-dipped galvanized steel. Plates shall be applied to both faces of truss at each joint. Where dimensions are not shown, place plates symmetrically about joint. Where no sheathing is applied directly to top chords, they shall be braced at intervals not exceeding 3'-0". Where no rigid ceiling is applied directly to bottom chords, they shall be braced at intervals not exceeding 10'-0". All additional lateral bracing specified on truss is for bracing individual truss members only. All permanent bracing for the overall structure is to be provided by designer of complete structure. TrusWal-Ronel bears no responsibility for the erection of trusses. Persons erecting trusses are cautioned to seek professional advice regarding temporary erection bracing which is always required to prevent toppling and "dominoing". This truss has been designed to meet applicable provisions of the "National Design Specifications for Stress-Grade Lumber and its Fastenings" (NFPA) and "Design Specifications for Light Metal Plate Connected Wood Trusses" (TPI). Cutting and Fabrication shall be accomplished using equipment which will produce snug fitting joints and plates. Care should be exercised at all times to avoid damage through careless handling of trusses during unloading, storing and erection.

LIVE LOAD	40	p.s.f.
DEAD LOAD	10	p.s.f.
CEILING L. L.		p.s.f.
CEILING D. L.	5	p.s.f.
	55	p.s.f.

NO Allowable Unit Stress Increase for Short Term Loading

TRUSWAL RONEL

JOB NAME TEST - F.H.A.		
PITCH/DEPTH 12"	SPAN 17'-4"	SPACING 24"
DRAWN BY CV	CHECKED BY	DATE 4/22/77
FILE NO. FTF 1217 TEST		

JOB NO. 760000 1 OF 1 PU-SHC 8-31-76 30

...FD IN ACCORDANCE WITH TPI-74 AND 1973 NDS

```
TYPE     = 2085                                      UNIFORM LOADING
SPAN     = 17FT. -  6IN.                             TOP CHORD LL =  40.0 PSF
SPACING  =  2FT. -  0IN. ON CENTER                             DL =  10.0 PSF
SHORT TERM LOADING INCREASES                         BOT CHORD LL =   0.0 PSF
   LUMBER STRESSES INCREASED = 1.00                            DL =  10.0 PSF
   PLATE  RATING  INCREASED = 1.00                   TOTAL LOAD   =  60.0 PSF
TOP CHORD SLOPE = 0.0000/12.
DISTANCE FROM LEFT END TO CENTER OF DUCT OPENING =  8 FT.  9IN.

HYDRO-AIR POSI-SPAN TRUSS                            OVER-ALL DEPTH = 12 IN
```

*** TRUSS IS SYMMETRICAL ABOUT THE CENTERLINE ***

PANEL	LENGTH	CHORD	FORCE	WEB	FORCE	JOINT	LOAD	REACT
P 1= 0FT	10- 8/16IN	C 1=	-1038	W 1=	1566	J 1=	11	-1049
P 2= 2FT	3- 3/16IN	C 2=	-1264	W 2=	-1409	J 2=	113	
P 3= 2FT	1-11/16IN	C 3=	-3176	W 3=	1060	J 3=	220	
P 4= 2FT	1-11/16IN	C 4=	-4452	W 4=	-992	J 4=	214	
P 5= 1FT	1-10/16IN	C 5=	-5172	W 5=	653	J 5=	163	
P 6= 2FT	1- 8/16IN	C 6=	-5172	W 6=	-585	J 6=	162	
P16= 2FT	1- 8/16IN	C16=	5172	W 7=	337	J17=	43	
P17= 2FT	2- 7/16IN	C17=	4905	W 8=	-162	J18=	43	
P18= 2FT	1-11/16IN	C18=	3945	W 9=	-0	J19=	42	
P19= 2FT	1-11/16IN	C19=	2355			J20=	33	
P20= 1FT	2- 6/16IN	C20=	0					

TOP CHORD IS 4X2 NO 2 KD DENSE SOUTHERN PINE — F=1850 T=1050 C=1350, STRESS FACTOR= 0.897

BOT CHORD IS 4X2 NO 2 KD DENSE SOUTHERN PINE — F=1850 T=1050 C=1350, STRESS FACTOR= 0.996

2X 4 MINIMUM BEARING REQUIRED AT J 1 AND J12

JOB NUMBER 760000 1 OF 1 PAGE 2

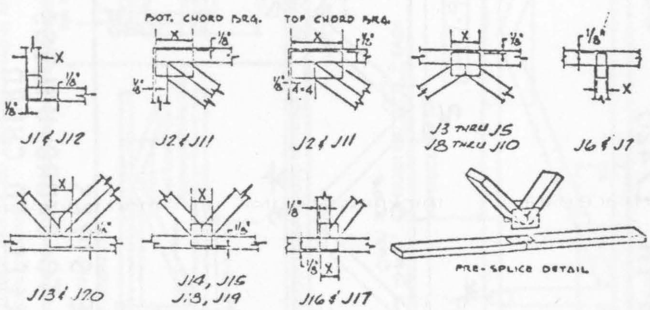
JOINT	POSI-TOOTH PLATE SIZE				** PLATE LOCATION ** ** X **	** Y **
J 1	1	X	3	PT	1IN	
J 2	3 1/2	X	6	PT	6IN	
J 3	3	X	4	PT	4IN	
SJ 4*	3	X	4	PT	4IN	
	PRE-SPLICE IS 3	X	5	PT		
J 5	3	X	4	PT	4IN	
J 6	1	X	3	PT	1IN	
J 9*	3	X	4	PT	4IN	
SJ14*	3	X	4	PT	4IN	
	PRE-SPLICE IS 3	X	5	PT		
J17	2 1/2	X	4	PT	4IN	
J18	3	X	4	PT	4IN	
J19*	3	X	4	PT	4IN	
J20	3 1/2	X	6	PT	6IN	

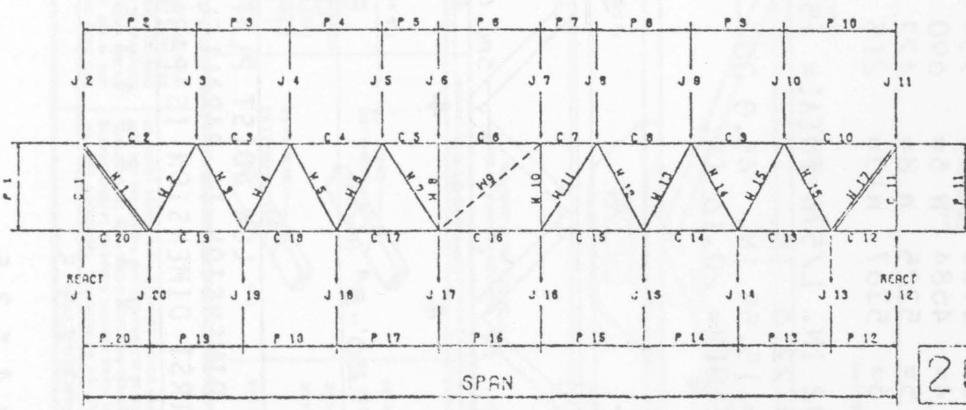
JOINTS MARKED "*" HAVE NO SYMMETRICAL MATCH
** OPTIONAL SPLICES **

PT= 20 GA. (210 PSI USING GROSS PLATE CONTACT AREA)

WEBS ARE TO BE 4X 2 NO 3 KD SOUTHERN PINE

THE FOLLOWING WEBS ARE DOUBLED 1, 17,
WEB 9 IS TO BE REMOVED





2085

NOTES:
1. CUT ALL MEMBERS TO BEAR.
2. CENTER ALL PLATES ON JOINTS UNLESS OTHERWISE NOTED.
3. INDICATED CHORD SPLICES SHALL BE LOCATED 1/4 OF THE PANEL LENGTH (±6") MEASURED FROM A JOINT.

TEST 0-5 | 20'-11" | 14" O.A.H. | FLOOR TRUSS

LUMBERMATE COMPANY
SAINT LOUIS, MISSOURI

TOP CHORD LIVE	40	PSF
DEAD	10	PSF
BOTTOM CHORD DEAD	5	PSF
TOTAL DESIGN LOAD	55	PSF
TRUSS CENTERS	2.00	FT.
UNIT STRESS INCREASE LUMBER	0	%
PLATES	0	%

LUMBER SPECIFICATIONS		F	T	C
TOP CHORD	2X4F SOU PINE #2 DN KD	1850	1050	1350
BOTTOM CHORD	2X4F SOU PINE #2 DN KD	1850	1050	1350
WEBS	2X4F SOU PINE #3 KD	875	525	700

REPETITIVE MEMBER BENDING STRESS USED IN THIS DESIGN

PLATE SERIES T GAGE 20 RATING 25.5 #/T: CAMBER L/720 KAS

THIS DESIGN SUGGESTION IS INTENDED FOR USE BY THE BUILDING ARCHITECT AND ENGINEER IN PREPARATION OF THEIR FINAL DESIGNS. NO RESPONSIBILITY IS ASSUMED FOR THE ERECTION, BRACING, AND ASSEMBLY TO THE COMPLETE STRUCTURE.

DESIGN BASED ON CRITERIA ESTABLISHED BY THE TRUSS PLATE INSTITUTE AND "NDS" BY THE NATIONAL FOREST PRODUCTS ASSOCIATION.

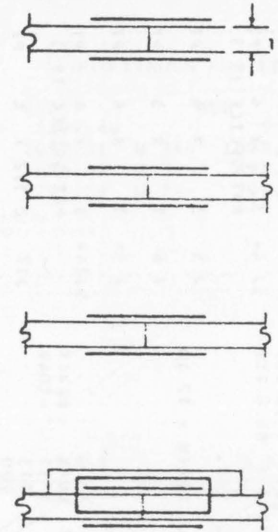
CUT MEMBERS TO BEAR. LATERALLY SUPPORT CHORDS.

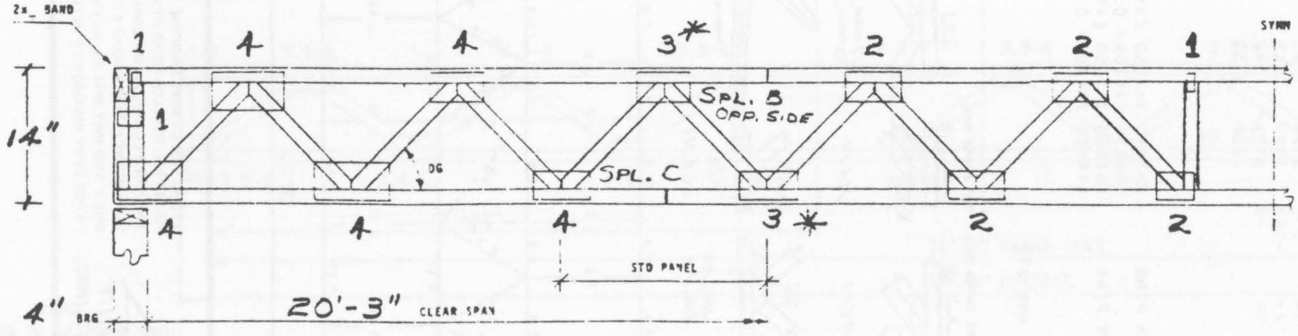
LUMBERMATE TRUSS PLATES OF GALVANIZED STEEL ARE INDICATED BY GAGE AND SIZE. PRESS PLATES SECURELY ON BOTH SIDES OF JOINTS. CENTER PLATES ON JOINTS UNLESS NOTED.

PANEL POINT LOADS AXIAL FORCES

T 1= 0	B 1= 932	W 0= -137	W 1= -1410	R 1= 1114
T 2= -1742	B 2= 2537	W 2= 1227	W 3= -1203	
T 3= -3170	B 3= 3788	W 4= 959	W 5= -934	
T 4= -4244	B 4= 4684	W 6= 690	W 7= -666	
T 5= -4963	B 5= 5225	W 8= 422	W 9= -398	
T 6= -5367	B 6= 5367	W10= 215	W11= -142	
T 7= -5367				

DEFL: LIVE= .42 IN. L/578 TOTAL= .58 IN. L/420
STD PANEL DIM= 22.0 IN.
20 STD WEBS= 15.55 IN. 45.0 DG
CENTER DAYLITE DIM= 20.16 IN.



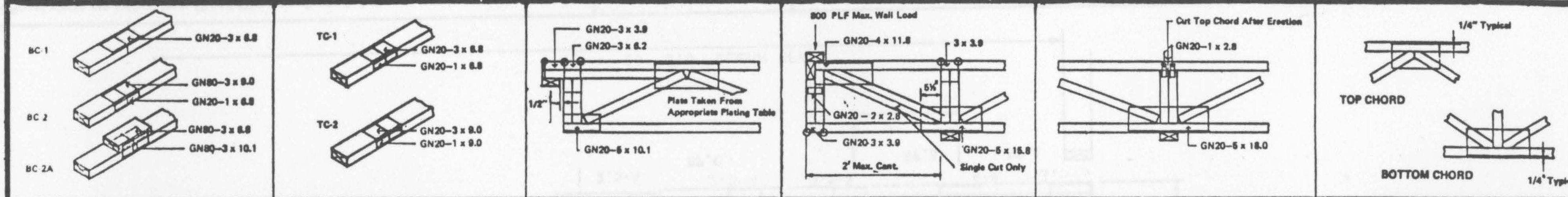


LOW JOIST PLATE SIZES
SECOND DIMENSION IS PARALLEL TO CHORD UNLESS NOTED
* - FIRST DIMENSION IS PARALLEL TO CHORD
1 = 1.5 x 3.5 4 = 4 X 5.3
2 = 3 X 3.5
*3 = 4 X 3.5

TRUSS PLATE INSTITUTE

LUMBERMATE® OPEN WEB L-O-W JOISTS® WOOD TRUSS FLOOR AND ... SYSTEMS

HANDLING AND ERECTION	MISCELLANEOUS INFORMATION	FABRICATOR:
CARELESS HANDLING OF TRUSS(ES) SHALL NOT BE PERMITTED. TEMPORARY AND PERMANENT BRACING FOR HOLDING TRUSS(ES) PLUMB AND FOR RESISTING LATERAL FORCES SHALL BE DESIGNED AND INSTALLED BY OTHERS. NO LOADS, OTHER THAN THE ERECTORS, ARE TO BE APPLIED TO TRUSS(ES) UNTIL AFTER ALL BRACING AND FASTENINGS ARE COMPLETED. AT NO TIME SHALL CONCENTRATED LOADS, GREATER THAN DESIGN LOADS, BE APPLIED TO TRUSS(ES).	THIS DATA SHEET, AND THE INFORMATION HEREON, IS THE PROPERTY OF AUTOMATED BUILDING COMPONENTS, INC., AND IS NOT TO BE COPIED IN WHOLE OR IN PART, OR USED FOR UNAUTHORIZED EXPLOITATION OF THE ITEMS DISCLOSED HEREIN, OR IN ANY OTHER WAY DISCLOSED OR USED FOR FURNISHING INFORMATION TO OTHERS. USE OF THIS STRUCTURAL COMPONENT IN A COMPLETE STRUCTURE MUST BE AT THE SPECIFICATION OF THE DESIGNER OF SAID COMPLETE STRUCTURE. ALL LATERAL BRACING SPECIFIED HEREON IS FOR BRACING INDIVIDUAL TRUSS MEMBERS ONLY. RESTRAINT OF LATERAL BRACING AND ADDITIONAL BRACING FOR OVERALL STRUCTURE IS TO BE PROVIDED BY DESIGNER OF COMPLETE STRUCTURE.	



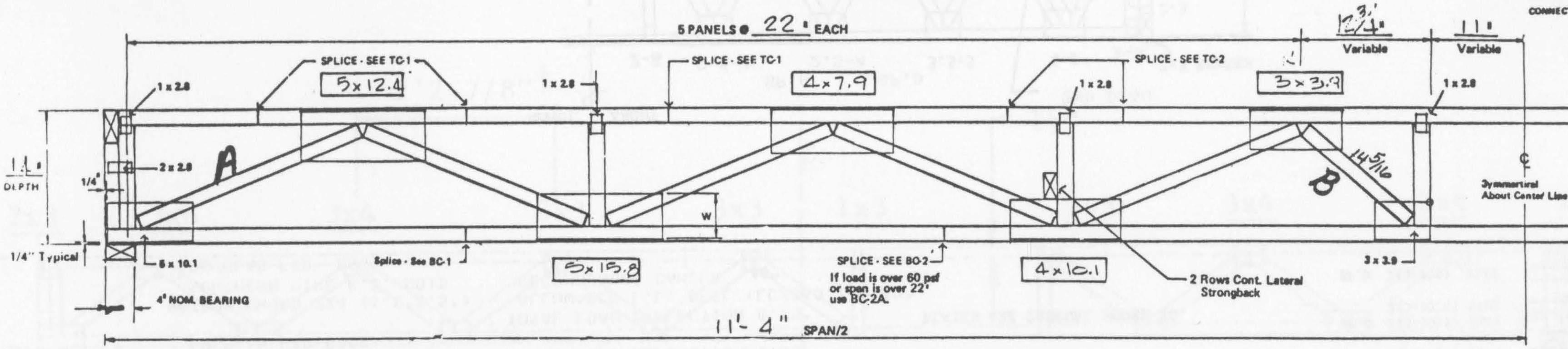
BOTTOM CHORD SPLICES	TOP CHORD SPLICES	ALTERNATE TOP CHORD BEARING	ALTERNATE CANTILEVER CONDITION	ALTERNATE INTERIOR BEARING CONDITION	ALTERNATE DOUBLE CUT WEBS

All plates GN20 except BC-2 & BC-2A

Note: Maximum one splice per chord on each side of centerline.

Note: See special plating instructions for Cantilever Trusses.

FOR USE WITH **GANG-NAIL®** CONNECTORS ONLY



Plates shown for Double Cut / Square Cut Webs

CAMBER (inches) = SPAN (in feet) / 40 = 5/8

*20 AND/OR 18

DESIGN & FABRICATION DATA	GANG-NAIL HARDWARE	LUMBER SPECIFICATIONS	DESIGN CRITERIA
DESIGN AND FABRICATION MEETS WITH LATEST REVISION OF: "NATIONAL DESIGN SPECIFICATIONS FOR STRESS-GRADE LUMBER AND ITS FASTENINGS" BY NFPA; "TIMBER CONSTRUCTION STANDARDS" BY AITC AND "DESIGN SPECIFICATIONS FOR LIGHT METAL PLATE CONNECTED WOOD TRUSSES" BY TPI.	STAMPED FROM . . . GA. GALVANIZED STEEL: MFD. & SUPPLIED BY AUTOMATED BUILDING COMPONENTS, INC., MIAMI, FLORIDA OR OUR SUBSIDIARIES. INSTALL GANG-NAIL HARDWARE ON BOTH SIDES OF TRUSS FACE.	TOP CHORD 4 x 2 NO. 1 K.D. SO. PINE EQUAL OR BETTER	TOP CHORD LIVE LOAD = 40 P.S.F. DEAD LOAD = 10 P.S.F.
	NOTE	BOTTOM CHORD 4 x 2 NO. 1 DENSE KD. SO. PINE EQUAL OR BETTER	BOTTOM CHORD LIVE LOAD = — P.S.F. DEAD LOAD = 5 P.S.F. TOTAL DESIGN LOAD = 55 P.S.F.
	POSITION PLATES SYMMETRICALLY ABOUT THE JOINT UNLESS OTHER DIMENSIONS ARE SHOWN. THIS "O" SYMBOL INDICATES THE POSITION OF PLATE IN RELATION TO WOOD MEMBERS.	WEBS 4 x 2 NO. 3 SO. PINE OR DOUG. FIR-LARCH EQUAL OR BETTER	TRUSS RAFTER SPACING = 2'-0" C/C. UNIT STRESS INCREASE FOR SHORT TIME LOADING: GANG-NAIL 0 % LUMBER 0 %. GN 20 32 LBS./NAIL, 227 LBS./IN.^{2} GN 80 — LBS./NAIL, — LBS./IN.^{2}

4x2 EconoFloor — Truss

SPAN = 22'-8" DEPTH = 14"

QUAKE 211

DESIGNED BY	DATE	JOB NUMBER
JN	4-15-76	B-3330
DRAWN BY	CODE	DRAWING NUMBER
EE	TPI-74	
APPROVED		
AMP		

JOB:FHA TEST TRUSS 21'-1" OVERALLL SPAN

SPAN	DEPTH	WEB <	LIVE L.	DEAD L.	CEILING	IN O.C.	BRG	DEF CODE
20' 6.0"	12.0"	33.7 DG	40.0 P S F	10.0 P S F	5.0 P S F	24.0"	1.0 KEY	360.0 DEF CON

NUMBER OF FULL PANELS AT 2.25 FT= 8

NOTE:TRUSS PLATED FOR SOUTHERN PINE

T/C		B/C		WEB		WEB		VERT	
T 1=	0.00	B 1=	1369.61	W 1=-	1735.35				
T 2=-	2500.73	B 2=	3631.86	W 2=	1433.18	W 3=-	1433.18		
T 3=-	4445.00	B 3=	5258.15	W 4=	1030.29	W 5=-	1030.29		
T 4=-	5753.32	B 4=	6248.50	W 6=	627.40	W 7=-	627.40		
T 5=-	6425.69	B 5=	6602.89	W 8=	224.51	W 9=-	224.51	V 1=	0.0

LUMBER SPECIFICATIONS
====================

TOP CHORD 2X4 (1.5"3.5")
SOUTHERN PINE K.D. NO1D
DOUGLAS FIR SSDF

BOTTOM CHORD 2X4 (1.5"3.5")
SOUTHERN PINE K.D. NO1D
DOUGLAS FIR DSS

DEFLECTION ANALYSIS SOUTHERN PINE
=================================

DEAD LOAD DEFLECTION 0.20"
LIVE LOAD DEFLECTION 0.55"

TOTAL LOAD DEFLECTION 0.75"
ALLOWABLE L.L. DEFL (LC/360.)=0.68"
RECOMMENDED CAMBER .40"

PATENT NO. 3651,612

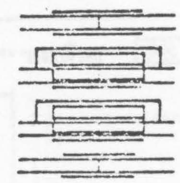
NOTES:
1) WEB MEMBERS 2x4 (1.5x3.5) STANDARD L.F. DOUGLAS FIR OR NO. 3 SOUTHERN PINE
2) PLATE SIZES ARE GIVEN AS WIDTH AND LENGTH AND SHOULD BE POSITIONED IN THAT MANNER UNLESS OTHERWISE NOTED.
3) SPLICES

SP.A 2(3-1018)

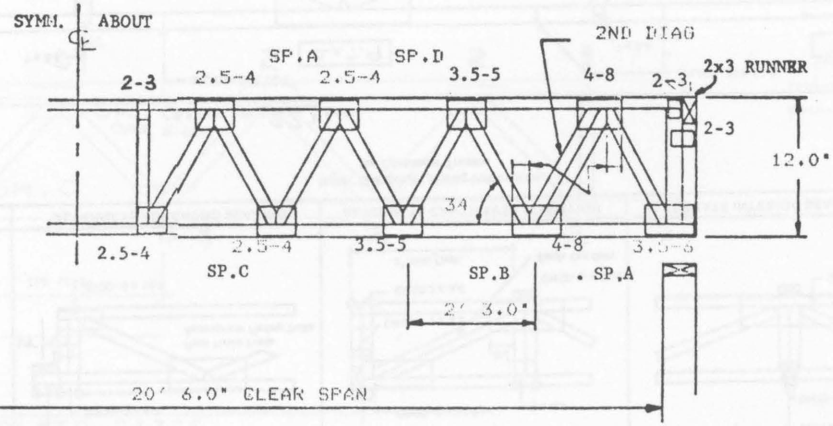
SP.B 2(3-1018) FLAT
2(2.5-10) SIDE

SP.C 2(3-1018) FLAT
2(3-1018) SIDE

SP.D 2(3-818) FLAT



PLATES ARE TRUSWAL MODEL 20.



TrusWal-Ronel plates are formed from 18 and 20 gauge, Grade A, hot-dipped galvanized steel. Plates shall be applied to both faces of truss at each joint. Where dimensions are not shown, place plates symmetrically about joint. Where no sheathing is applied directly to top chords, they shall be braced at intervals not exceeding 3'-0". Where no rigid ceiling is applied directly to bottom chords, they shall be braced at intervals not exceeding 10'-0". All additional lateral bracing specified on truss is for bracing individual truss members only. All permanent bracing for the overall structure is to be provided by designer of complete structure. TrusWal-Ronel bears no responsibility for the erection of trusses. Persons erecting trusses are cautioned to seek professional advice regarding temporary erection bracing which is always required to prevent toppling and "dominoing". This truss has been designed to meet applicable provisions of the "National Design Specifications for Stress-Grade Lumber and its Fastenings" (NFPA) and "Design Specifications for Light Metal Plate Connected Wood Trusses" (TPI). Cutting and Fabrication shall be accomplished using equipment which will produce snug fitting joints and plates. Care should be exercised at all times to avoid damage through careless handling of trusses during unloading, storing and erection.

LIVE LOAD	40.0	p.s.f.
DEAD LOAD	10.0	p.s.f.
CEILING L. L.		p.s.f.
CEILING D. L.	5.0	p.s.f.
	55.0	p.s.f.

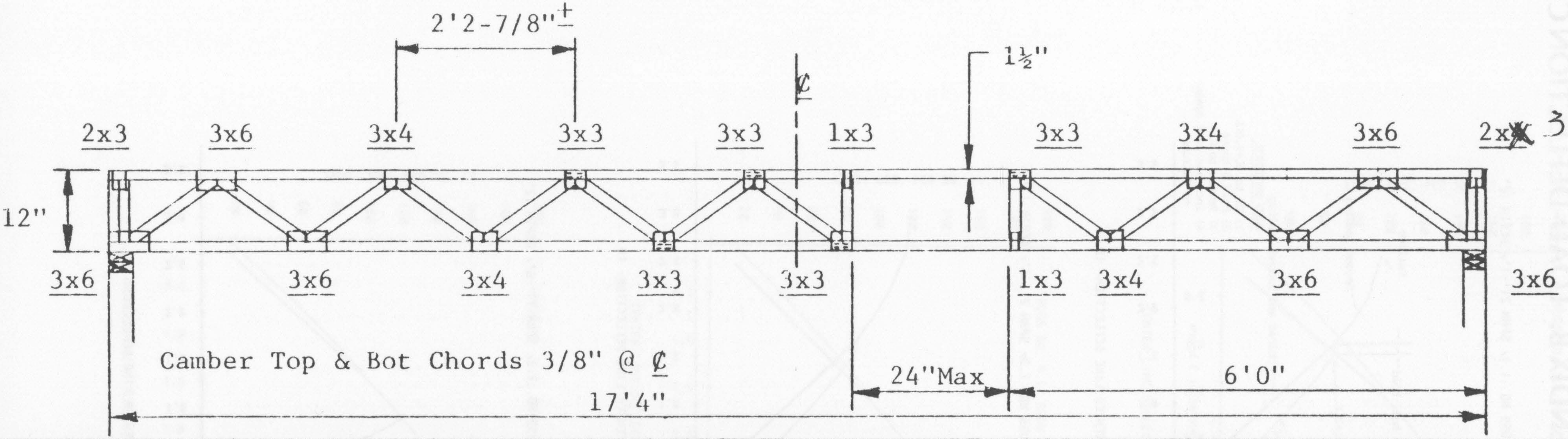
0.0 PC

Allowable Unit Stress Increase for Short Term Loading

TRUSWAL RONEL

JOB NAME	FLORTRUS	
PITCH/DEPTH 12.00 IN	SPAN [illegible]	SPACING [illegible]
DRAWN BY [illegible]	CHECKED BY	DATE [illegible]
FILE NO. FTR 12.0 20		

Top Chord : 2x4 #1 Dense KD Southern Pine
Bot Chord : 2x4 #1 Dense KD Southern Pine
Webs : 2x4 #3 Southern Pine
No wane or loose knots should occur in the plate contact area.



ALPINE TRUSS

GENERAL NOTES

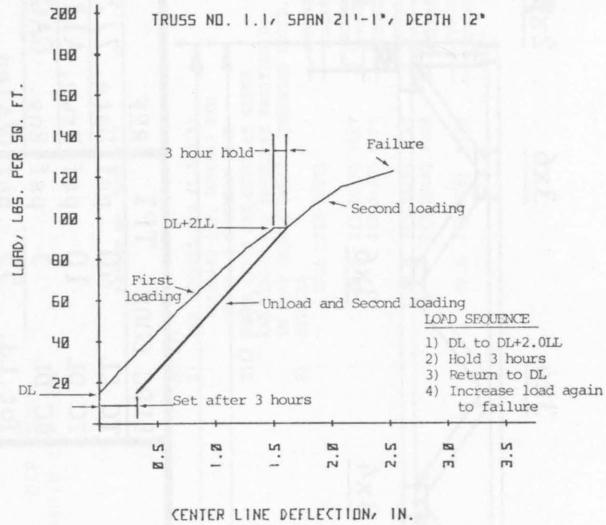
ALPINE CONNECTORS ARE 20 GAGE GALVANIZED STEEL AND SHALL BE APPLIED TO BOTH FACES OF TRUSS AT EACH JOINT. PLATES SHALL BE LOCATED AS SHOWN ABOVE. A TOLERANCE OF UP TO 10% OF GIVEN PLATE AREA ON ANY MEMBER IS PERMITTED. UNLESS THE ABOVE SPECIFICATIONS FOR LUMBER AND ALPINE CONNECTORS ARE FOLLOWED, THERE SHALL BE NO WARRANTIES OF THIS DESIGN, EXPRESS OR IMPLIED. CUTTING & FABRICATION REQUIRES EQUIPMENT WHICH WILL PRODUCE SNUG-FITTING JOINTS AND PLATES. SEE "QUALITY CONTROL MANUAL" BY TRUSS PLATE INSTITUTE (TPI). OVERALL LENGTHS ASSUME 4" BEARING EACH END. BEARING WIDTHS UP TO 8" MAY BE USED, PERMITTING AN INCREASE IN OVERALL LENGTH UP TO 8". DESIGN STANDARDS CONFORM WITH APPLICABLE PROVISIONS OF "NATIONAL DESIGN SPECIFICATION FOR STRESS GRADE LUMBER AND ITS FASTENINGS" (NATIONAL FOREST PRODUCTS ASSOCIATION) AND "DESIGN SPECIFICATIONS FOR LIGHT METAL PLATE CONNECTED WOOD TRUSSES" (TPI).

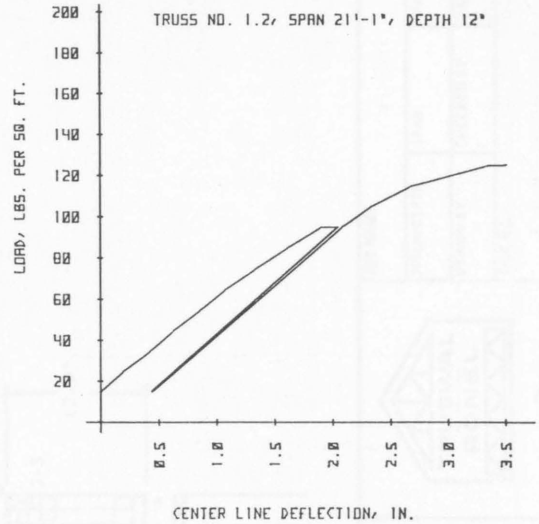
WARNING

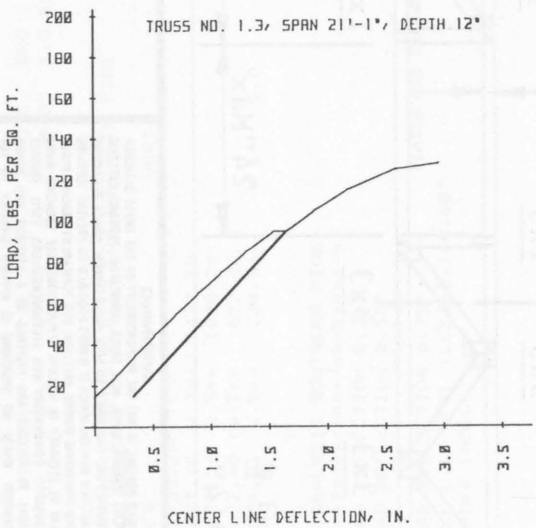
FIELD BRACING IS NOT THE RESPONSIBILITY OF THE TRUSS DESIGNER, PLATE MANUFACTURER, NOR TRUSS FABRICATOR. PERSONS ERECTING TRUSSES ARE CAUTIONED TO SEEK PROFESSIONAL ADVICE REGARDING ERECTION BRACING WHICH IS ALWAYS REQUIRED TO PREVENT TOPPLING AND DOMINOING DURING ERECTION, AND PERMANENT BRACING WHICH MAY BE REQUIRED IN SPECIFIC APPLICATIONS. SEE "BRACING WOOD TRUSSES": COMMENTARY AND RECOMMENDATIONS (TPI). TRUSSES SHALL BE ERECTED AND FASTENED IN A STRAIGHT AND PLUMB POSITION. WHERE NO SHEATHING IS APPLIED DIRECTLY TO TOP CHORDS, THEY SHALL BE BRACED AT A MAX. SPACING OF 3'0" O.C. WHERE NO RIGID CEILING IS APPLIED DIRECTLY TO BOTTOM CHORDS, THEY SHALL BE BRACED AT A MAX. SPACING OF 10'0" O.C. TRUSSES SHALL BE HANDLED WITH REASONABLE CARE DURING FABRICATION, SHIPPING, AND ERECTION TO PREVENT DAMAGE.

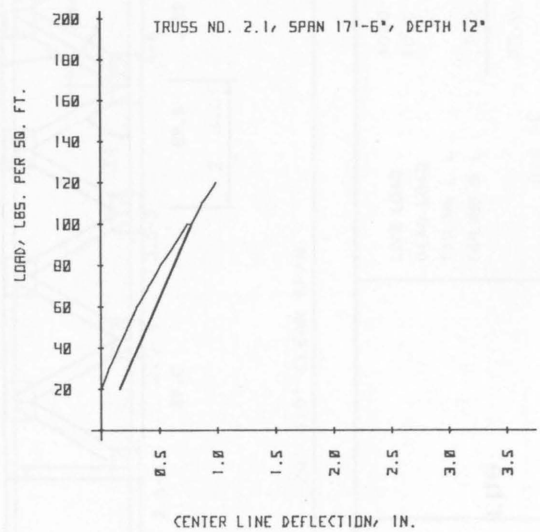
BLDG CODE	TPI		REF	
TC LL	40	psf	Date	7/28/77
TC DL	10	psf	Drwg.	A114,733
BC DL	5	psf	Eng.	GAG/OAG
Tot.Ld.	55	psf	O/A Len.	17'4"
Dur.Fac.	1.00		Depth:	12"
Spacing	24.0"		Type	SYS 42 F

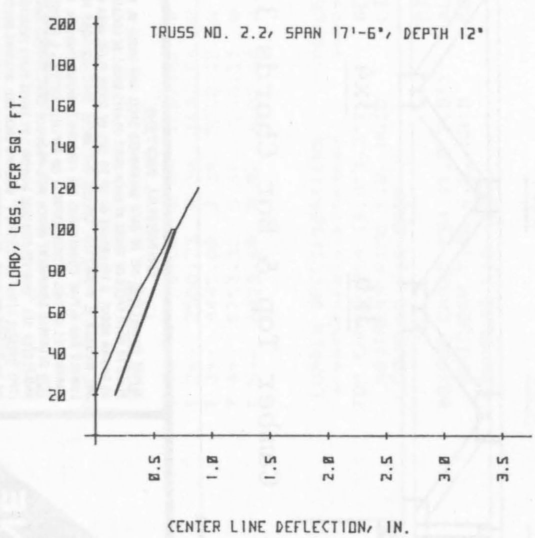
APPENDIX B. LOAD-DEFLECTION CURVES FOR SHORT-TERM TESTS

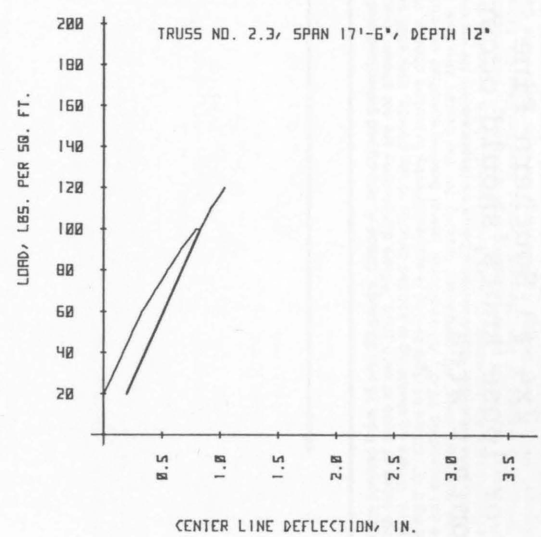


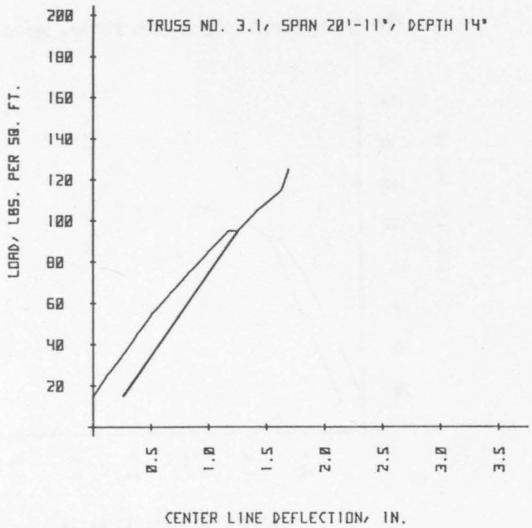


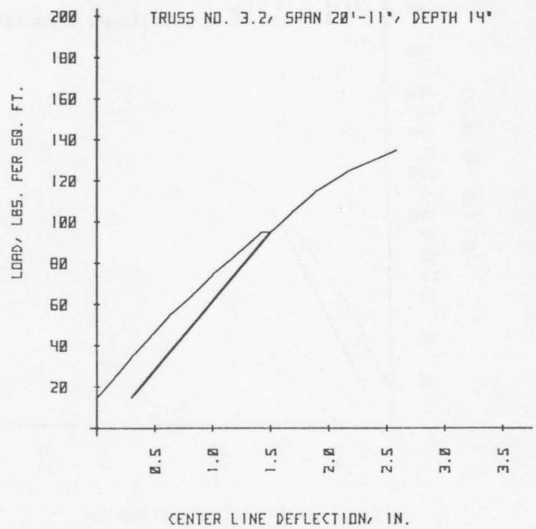


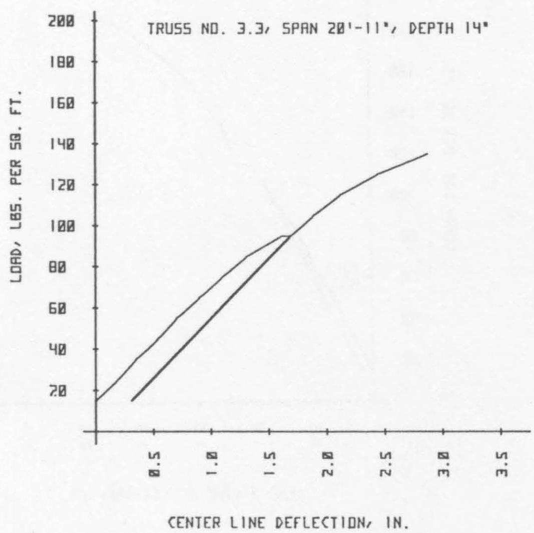


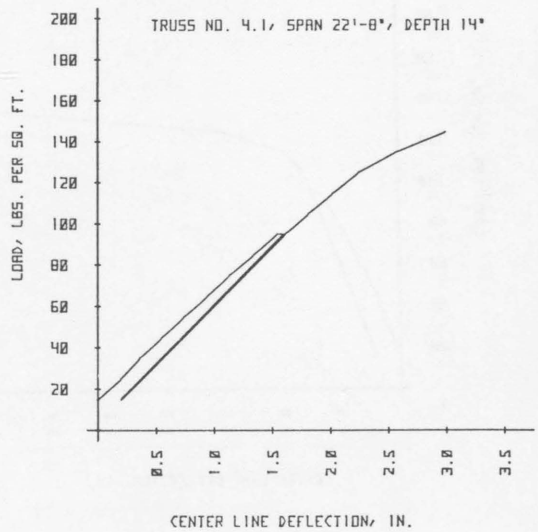


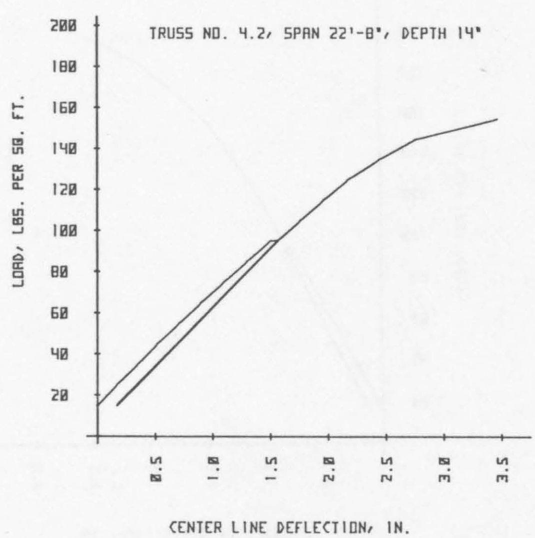


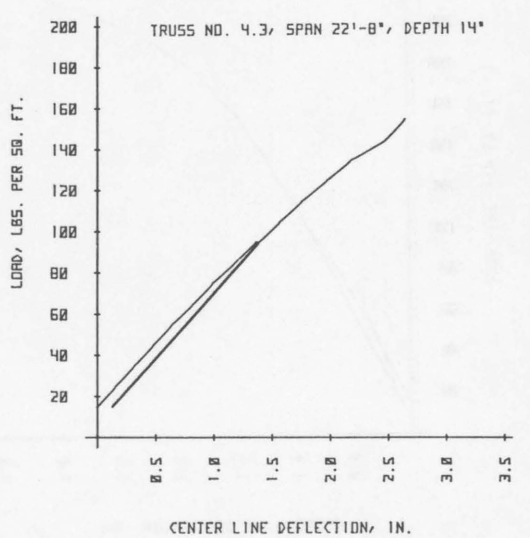

TRUSS NO. 3.1, SPAN 20'-11", DEPTH 14"
LOAD, LBS. PER SQ. FT.
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.

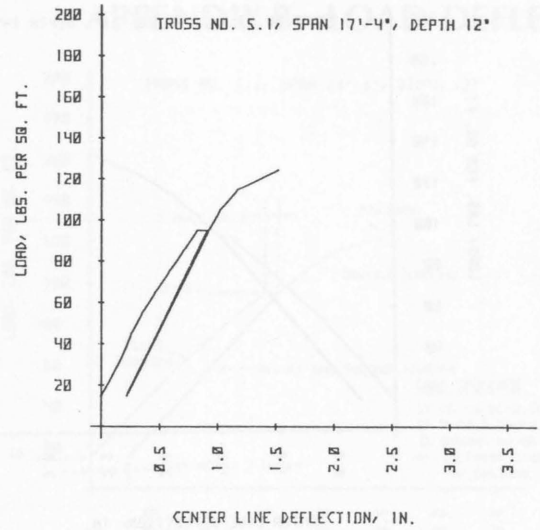

TRUSS NO. 3.2, SPAN 20'-11", DEPTH 14"
LOAD, LBS. PER SQ. FT.
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.

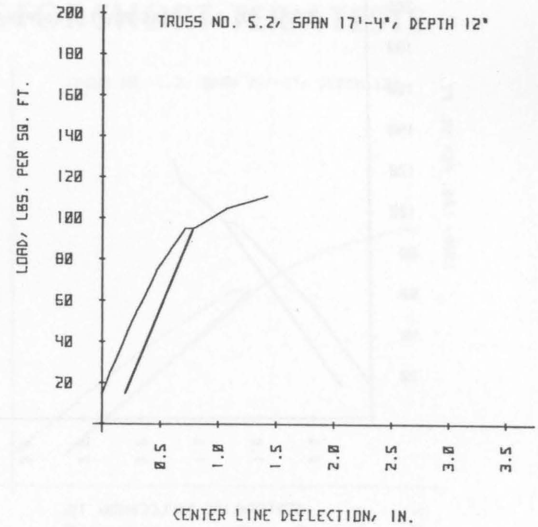

TRUSS NO. 3.3, SPAN 20'-11", DEPTH 14"
LOAD, LBS. PER SQ. FT.
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.

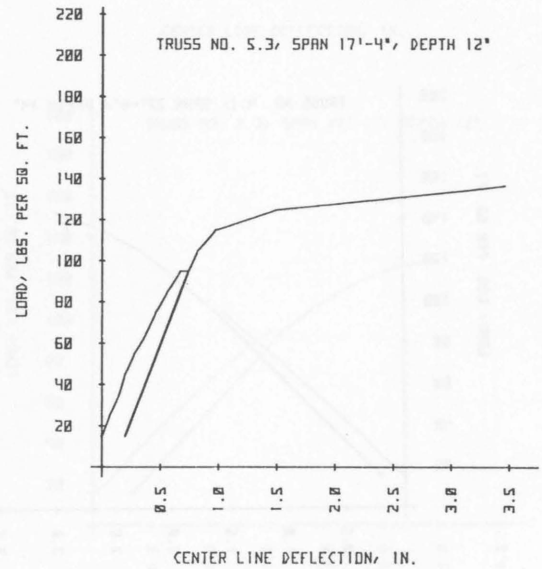

TRUSS NO. 4.1, SPAN 22'-8", DEPTH 14"
LOAD, LBS. PER SQ. FT.
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.

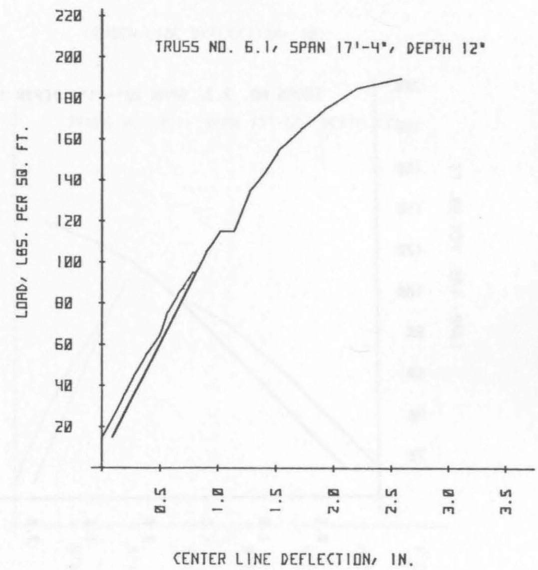

TRUSS NO. 4.2, SPAN 22'-8", DEPTH 14"
LOAD, LBS. PER SQ. FT.
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.

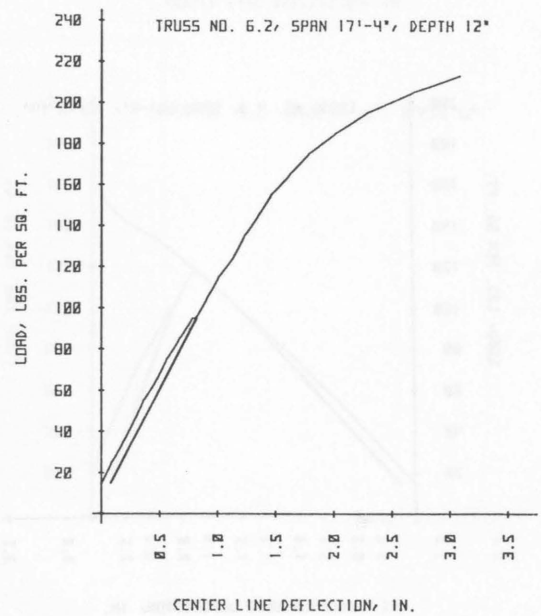

TRUSS NO. 4.3, SPAN 22'-8", DEPTH 14"
LOAD, LBS. PER SQ. FT.
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.

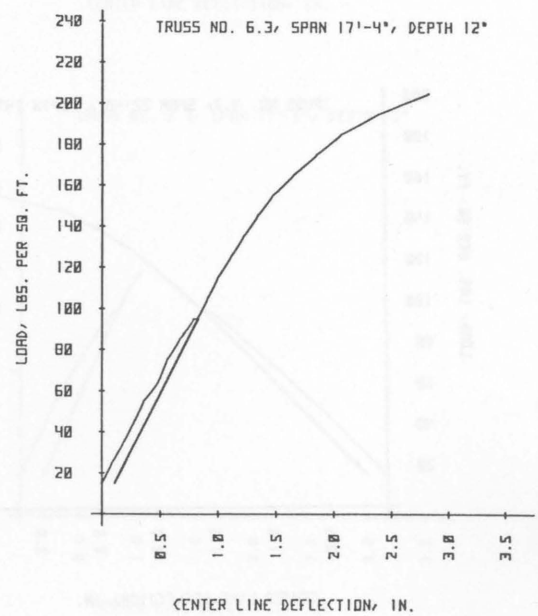

TRUSS NO. 5.1, SPAN 17'-4", DEPTH 12"
LOAD, LBS. PER SQ. FT.
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.


TRUSS NO. 5.2, SPAN 17'-4", DEPTH 12"
LOAD, LBS. PER SQ. FT.
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.


TRUSS NO. 5.3, SPAN 17'-4", DEPTH 12"
LOAD, LBS. PER SQ. FT.
220
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.


TRUSS NO. 6.1, SPAN 17'-4", DEPTH 12"
LOAD, LBS. PER SQ. FT.
220
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.


TRUSS NO. 6.2, SPAN 17'-4", DEPTH 12"
LOAD, LBS. PER SQ. FT.
240
220
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.


TRUSS NO. 6.3, SPAN 17'-4", DEPTH 12"
LOAD, LBS. PER SQ. FT.
240
220
200
180
160
140
120
100
80
60
40
20
0.5
1.0
1.5
2.0
2.5
3.0
3.5
CENTER LINE DEFLECTION, IN.